EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION

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Berkeley, California
July 11-15, 1977

In Three Volumes

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Preface

The material contained in these three volumes constitutes the proceedings of a workshop on Earthquake-Resistant Reinforced Concrete Building Construction (ERCBC) sponsored by the National Science Foundation, and held at the University of California, Berkeley, July 11-15, 1977. The main purposes of the workshop were to provide a means for the exchange of information related to the state-of-the-art and state-of-the-practice in the design and construction of seismic-resistant reinforced concrete buildings, to evaluate current progress, and to establish research needs and priorities for future work.

The specific objectives and organization of the workshop are summarized in the Introduction to the first volume. The final recommendations of the workshop form the main body of that volume. Four appendixes follow, containing the program, the list of participants, the list of working groups, and, lastly, a research directory.

Volumes 2 and 3 comprise the technical reports and papers that were presented. These furnished the background material for the discussions which ultimately resulted in the final recommendations of the workshop.

It is hoped that these proceedings will help mitigate the destructive effects of earthquakes by encouraging practitioners to implement those recent findings from the research and professional communities that will improve current practice in ERCBC, and by providing researchers and agencies sponsoring research with guidelines for ensuring that future research is oriented toward solving current problems. It is also hoped that the proceedings will serve to stimulate communication and improve cooperation between practitioners, educators, researchers, and representatives from industry and government agencies working in the field of ERCBC.

It is not possible here to thank all the individuals who contributed to the success of the workshop, but a few should be mentioned. The assistance of Dr. John B. Scalzi, Manager of the Earthquake Engineering Program of the National Science Foundation, during the planning of the workshop, and his continuous support and encouragement are gratefully acknowledged. The able assistance of Dr. Stephen A. Mahin, who acted as organizing secretary, throughout all phases of the workshop is greatly appreciated. In addition, thanks must be extended to the members of the steering committee: W. Gates, N. Hawkins, J. Scalzi, M. Sozen, and L. Wyllie, Jr., for their technical assistance; to the session chairmen; the heads and recording secretaries of the working groups; to H. Barry and L. Reid of University Extension for coordinating schedules, arranging accommodations, and making the workshop an enjoyable experience for all the participants; and to L. Tsai, not only for invaluable editorial assistance in the preparation of these volumes, but for her continued help throughout the various phases of the workshop. Finally, special and sincere appreciation goes to the authors of the technical reports and to all the participants, who took time from their busy schedules to collaborate in the workshop. The success of the workshop is the result of their individual and combined efforts.

Funding for this workshop was made possible by grant ENV76-01923 from the National Science Foundation. Their support is gratefully acknowledged. These proceedings constitute the final report to the sponsor. The conclusions and recommendations expressed herein do not necessarily reflect the views of the National Science Foundation.

Vitelmo V. Bertero
Berkeley, California
June 1978

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED
CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

AN OVERVIEW OF THE STATE-OF-THE-ART IN EARTHQUAKE RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION IN THE UNITED STATES OF AMERICA

by

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INTRODUCTION

Much has been accomplished in the United States, and elsewhere, toward the improvement of procedures in design and construction leading to the greater and more reliable resistance of reinforced concrete buildings to severe ground shaking caused by earthquakes. However, it would be unwise for me in this overview paper to attempt to catalog all that has been done in the United States. There are too many efforts, and any listing would be certain to have too many omissions. The discussion of accomplishments will therefore be held to a few major categories and be somewhat general rather than detailed treatment which will no doubt follow in papers by others. Apologies are extended in advance for worthy efforts not included.

It should be noted before proceeding that the state-of-the-art in design -- not only as practiced but as generally recognized, has depended strongly upon whether one was concerned with nuclear plants, other "exotic" structures, or more conventional structures such as office buildings. Certain matters are being considered at this time for buildings of a normal-code type that have been in practice in the nuclear field for 10 or more years. Other things are being approached or considered in the nuclear field that are so new they haven't yet reached the university research level, let alone the design of conventional buildings. Another lesser known field in which the state-of-the-art has been well beyond normal practice or general knowledge is that related to the prediction and measurement of response to ground motion from underground nuclear or other explosions. The writer has been continuously and actively engaged in these various areas since their inception as well as in research; the following discussion will no doubt be influenced accordingly.

One should distinguish between the existence of knowledge or a state-of-art and whether or not that knowledge or art needs to be, or should be, applied in all cases. It is not expected, for example, that all of the refinements of nuclear plant design be applied to the design of all other buildings; nevertheless, the mere existence of knowledge eventually has some impact on all design, although there may be a long period of filtration or ingestion. Perhaps one of the main purposes of a meeting like this is to accelerate the exchange of such information.

Following certain general considerations there will be a brief discussion of significant accomplishments, then discussion of specific areas of accomplishment and of research needs in two broad categories -- those items that apply to buildings of any type or material, and those items which apply exclusively to the design or construction of reinforced concrete buildings. Although treatment of the first category will be limited in favor of the more specific one, for obvious reasons, it would be derelict not to consider both in this overview. For example, an improvement in knowledge of soil-structure interaction is an accomplishment that may improve the design of future concrete buildings as well as all others. In order to make the presentation as coherent as possible, both accomplishments and needs in research and development will be discussed under each subject.

PRELIMINARY GENERAL CONSIDERATIONS

Whenever the writer attends a meeting, or reads a summary of a meeting, on the subjects of the state-of-the-art and what we don't know, what research should be done, etc., his initial reaction is astonishment at how much we don't know, or think we don't know. His second reaction is that in the face of such ignorance, how could we have done what we have, or are all the products of our past efforts bad -- are they all potential earthquake failures? Some would reply that maybe most of the structures are all right but they were overdesigned, or cost too much. But that isn't a satisfactory reply because building construction today is enormously costly on a relative basis whether cost is measured in current dollars, 1930 dollars, chicken eggs, or glass beads!

His third reaction to the array of "ignorance" tabulated at such meetings is: maybe some of this is generated by the need for, or desire for, research subjects and supporting funds. Although this may be a factor in some cases, consciously or otherwise, it is not a dominant one. What, then, is the dominant factor -- how could we know so little and have done so much?

The answer seems to lie in the _way_ we are doing things today as compared to how they were done before. Today we are:

1. Doing things better, in general; and by "better", let us say more reliably.

2. We are using computers, and thus need more formal documentation.

3. We are using less judgment in favor of more hard numbers.

The more we use computers, the more precise we have to be in our criteria and instructions. We use less judgment in design today because (a) we rely more on computers; (b) because new generations brought up on computers may have less judgment to exercise (the reason for this being that judgment is not required so much and therefore is less developed); and (c) because society through its legal structure and current practice is making it risky to rely too much upon judgment where there are other means. This explains why we have professed ignorance today -- and have developed long research lists -- in subjects which have in a great many cases been treated before, and often in a quite competent manner.

My maternal grandfather was a country doctor who did some marvelous things for his patients -- things he couldn't do if alive today because: (a) he would be sued; (b) he would need to consult a bevy of specialists; and (c) the specialists would probably want extended, double blind, computerized research studies before reaching a conclusion; and (d) then AMA and perhaps governmental agencies would have to concur in the validity of the research and in the application of the findings.

Many subjects considered new or pressing research material today were treated years ago by good engineering judgment and intuition, or perhaps by a meagerly-funded research project. For example, the matter of foundation "tipping" as a possible mitigating factor under lateral forces was considered and used extensively in design decades ago [1], and yet today this is a viable, worthwhile subject that must be formalized [2]. Energy demands and ductility are similar [3,4]. Today one has to follow the book whereas an experienced engineer a few decades ago could do marvelous things even without the computer. Today, we are "ignorant" only in the sense that we must -- for various reasons -- be more rigorous even in things we may know much about. On this basis there is no objection to doing detailed research on our so-called "ignorance" providing, of course, that excessive duplication of effort is avoided. It would be desirable in the process, however, to give credit to more of the things done by earlier workers who operated, necessarily, in a different climate.

The main design aspects for an advanced state-of-the-art have been charted by Bertero [5] and are shown in Figure A. As complex as this may appear, there are even more aspects that should be considered either directly or implicitly. Some of these have to do with the many factors that may (or may not) reconcile the great range between design coefficients for base shears and the peak (instrumental) accelerations that may be expected. The writer has listed 22 such factors in a purposely provocative manner [6], some of which are shown in Figure A. There will be more and more consideration of these factors as time goes on. Detailed work has been done in some of them. Many of them can best be approached by realistic probabilistic analyses rather than deterministic procedures that may be unrealistically conservative.

A chart similar to Figure A for the construction aspects could be made but it would be very complex because of the many various ways in which the construction industry operates. A set of plans and specifications for a building are subject to various reviews after the design is done. Reviews of various intensities and for various purposes may be made by the owner, by building officials, by environmental and other agencies, by safety agencies, by fire departments and by financing institutions. The output from all this, after any necessary revisions of the documents, is a building permit, financing, and other necessary permits.

Perhaps simultaneously, a general contractor is being (or has been) obtained by various procedures including bidding, selection, and negotiation. There may or may not be a construction program manager. Often, and desirably, these people are available during the design stage. If not,

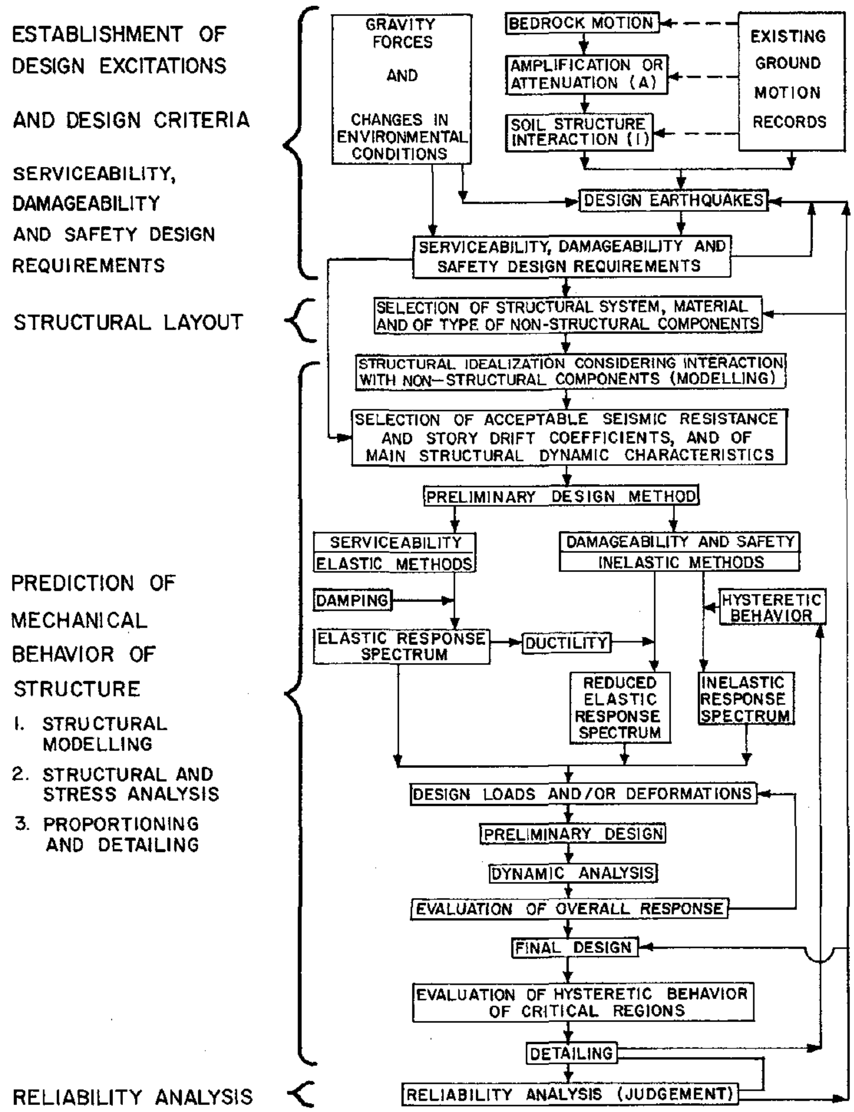

FIG. A - FLOW DIAGRAM OF GENERAL ASPECTS AND STEPS INVOLVED IN SEISMIC-RESISTANT DESIGN (Ref. 5) they may have some suggestions for alternative procedures. The later these surface in the process, the more costly changes may be.

The general contractor has various subcontractors and material suppliers. Obviously, the rebars and the concrete supplied are vital to the success of the design; their proper placement is also vital. A testing laboratory is involved to make both field and laboratory tests, to design mixes, submit reports, etc. In addition, an inspector (or more) is needed -- a very important position. The engineer has to approve all shop drawings, guide the inspection and lab team, and in general assure himself on a current basis that the job is going as intended and that there are no unforeseen problems going unattended. He no doubt has to certify that the building is constructed as shown on the contract documents.

All in all the construction of a modern reinforced concrete building is a major undertaking, one with all sorts of participants and responsibilities. Research does not belong in this stage, nor even in the design stage except to insure the proper practice of the state of the art. Researchers, however, should have some knowledge of the complexities, costs and problems in creating a modern building.

The ultimate goal of ERCBC should be to design and construct reliable buildings which will insure that in the event of an earthquake there will be no serious injury or loss of life and that, on the average, the cost of repair of earthquake damage will not exceed the increased design, construction, maintenance and financing costs that would have prevented the damage [7]. This can only come about with integrated good research, good building codes, good design and good construction.

SOME SIGNIFICANT ACCOMPLISHMENTS

Accomplishments in the field of earthquake engineering applied to reinforced concrete buildings, as in many other fields, are generally somewhat evolutionary and gradual. There are not dramatic discoveries such as finding a new element or the release of a new wonder drug. Even though the younger generations may not agree, a great deal of a basic nature about the earthquake problem was known decades ago. It may not have been fully demonstrated because of the sparsity of data and of research funds, nor was it generally recognized (most simply didn't care), but it was known. In the last 10 or more years much progress has been made in obtaining more and badly needed data whether from earthquakes per se, from testing, or from analysis, especially in the inelastic range. There has also been much progress in the dissemination of data and the transfer of technology to a greater population of engineers and building officials.

Decades ago, California structural engineers who were on their toes earthquake-wise provided "extra" closely spaced column ties near joints and "extra" closely spaced stirrups in beams or girders near columns. A few even put some in the joints. They also provided bar laps in an amount and at places that shocked eastern engineers. And they did other things they knew, or felt, were proper for earthquake resistance. ACI standards in those days did not require, or even recognize, such things. How did these California engineers know that such details were vital in earthquake resistance? They observed, they thought deeply, and -- above all -- they had the "feel of structures", a sixth sense that may be natural or developed, or both. Unfortunately, the old eastern (non-seismic) practice was followed in many other countries and subsequent earthquakes have led to damage or failure "even though designed to U.S.A. standards." I think this matter has been clarified and remedied once and for all and, if it has, I consider it a great accomplishment of recent times.

Perhaps the most significant, and certainly the most dramatic, item in the earthquake resistance of reinforced concrete construction is the combined concept and detailed procedures of making concrete "ductile" and to avoid disastrous frame failures in shear or unconfined compression during earthquakes. This was first published in a book by Blume, Newmark and Corning [8,9]. Unfortunately, it took several years for ductile concrete to get into earthquake codes (it was challenged by the steel industry) during which time many frames of a non-ductile nature were constructed. Although the book was released some 16 years ago, valuable work is still being conducted in the great inelastic range of ductile reinforced concrete whose properties can mean success or failure in a great earthquake. It is noted that a few older buildings tend to have some ductile characteristics because of accident or, more likely, because of the wisdom, or "feel", of the designers.

Another accomplishment in the last 10 years or so has been to test full size or large scale members, joints and assemblies in such manner as to better understand the properties and mechanics of reinforced concrete in the range from yield to ultimate strength, to increase ductility and energy absorption capacity, and to find ways of increasing the ductility/cost ratio. A great deal of such work has been done at UC Berkeley [10], not to mention Illinois [11], and the Portland Cement Association [12]. (Also see Appendix A)

Another accomplishment has been testing and analysis of so-called shear walls, alone and in various combinations with frames or boundary members [12]. The integration of the shear wall, or "cantilever" or "flexural" wall in many cases, as a full-fledged structural element rather than as a filler wall or as a "stiffener" in a building is a step forward. The action of coupled walls is an important phase of this subject. There have been a few tests of real buildings or test structures, some into the destructive range, and these have provided valuable information [13,14].

Foundation design has improved greatly as have considerations of soil-structure-interaction, another subject in which Seed and others at Berkeley have done much work. (See Appendix A)

There have been improvements in the joining of precast, prestressed, and poststressed elements for earthquake resistance. There have also been improvements in the placing of cast-in-place concrete.

There have been improvements in the method of joining or splicing bars and in detailing and placing reinforcing bars. Many designers have learned that a bar has other dimensions than length, and that the corrugations also occupy space! The construction people will appreciate this advance.

There have been more needed determinations of damping and stiffness with changes in distortion and into the inelastic range.

Yes, there have been accomplishments but there is much more to be done, as will be discussed subsequently.

SUBJECTS APPLICABLE TO ALL BUILDINGS

Ground Motion

Much more is known about ground motion characteristics, amplitudes, statistics and probabilities than was the case 10 years ago. In general, it has been found that peak ground accelerations (a) may be greater than was thought several years ago; (b) they are quite variable from place to place [13]; (c) they are not good indicators of structural response because the time dimension is not included; and (d) they have very weak correlation with magnitude close to the source [15].

The same problems exist, possibly to a lesser degree in certain frequency bands, for peak ground velocity and displacement. By themselves, as for acceleration, they do not constitute adequate descriptions of motion without the time dimension in the form of period and often duration as well. The relationships of ground acceleration, velocity and displacement, are also quite variable although they have been idealized.

Spectral response diagrams, when properly developed to conform to the given conditions and to allow for the probabilistic aspects of the problem, are quite useful and meaningful. So-called "standard" response spectra should be used with care and with proper attention to the stated limitations and conditions for such curves [16]. The zero-period, or "anchor" acceleration used to construct or to proportion response spectra must be carefully selected to avoid extreme conservatism. There will be much more discussion in the future of so-called "effective" acceleration as compared to "instrumental" acceleration [6,17].

Work is underway on the matters of seismic moment and stress drop. The results in a few years might well weaken magnitude as a parameter in earthquake engineering along with peak particle ground motion.

Soil-Structure Interaction

Much progress has been made by Seed and others in the matter of how the soil over rock affects the input motion [10]. There can be no doubt that the soil and the structure constitute the real dynamic system.* Much more needs to be done, however, on how large or deep foundations affect

*Even this was studied a long time ago [24,25].

the motion. Work of this type is underway now [18] and some papers have been published [19,20,21]. Some of the limitations of popular procedures is that they do not, at least adequately, allow for incident waves at various angles, for radiation from boundary layers assumed in analysis, or for the mitigating effects of large, rigid structures. Moreover, more needs to be done on the inelastic properties of soils and rock under various strain levels and cycling and on the true rigidities of foundations (and structures) vertically relative to the soil.

Sophisticated analyses can be made today, where needed. Some of the most elementary aspects of these are being injected into building codes [17,22]. It is expected that much further progress will be made in soil-structure interaction and in structure-soil-structure interaction in the next few years. It is also expected that although quasi-resonant response of the soil and structure is, of course, to be expected in some cases [23], it will be found, in general, that current procedures are conservative in (a) taking worst or envelope conditions, and (b) neglecting important energy dispersions and work done. As more exotic methods are developed, such matters as torsion and tilting with symmetric structures will have to be considered. Further work should also be done on two and three dimensional aspects of the ground motion and the response and the probabilistic aspects of dimensional and modal combinations and on the combined inelastic structure-inelastic-soil system.

Experimental research in the laboratory and especially in the field is needed to help resolve many problems in soil-structure interaction. The tipping tendency of tall slender structures is a viable subject [2] that needs to be considered more formally than it was in the past.

Theory

Theory of dynamic elastic superstructures has been advanced for a long time. The problem is to be able to model the prototype realistically. Real buildings are often quite complex with various interacting materials and elements, with various properties and participation at different distortion levels. A "non-structural" element may seriously affect the properties that control the response and thus be an important element until it fails. It is well known that the natural periods of many buildings change with distortion, with damage, and with the history of prior responses [13]. Analyses that do not allow for probable major period variations may not be realistic. The writer has often said, and repeats again, if one can't model the building the way it is, then the building should be made to conform to the model. In the normal climate for building design it is difficult, if not impossible, for the engineer to conform to the model.

More needs to be done in inelastic modeling, analysis and testing and also in 2D and 3D analysis elastically and in the inelastic range.

Techniques for energy reconciliation in design have been available a long time [3,8,26] but have not actually been proved or disproved. Formal proof may be a long time away. In spite of this, the balance of kinetic energy with elastic potential energy and the work done in the inelastic range seems not only logical but necessary and it is intuitively acceptable to most engineers. The reserve energy technique [3,8] is a powerful tool for complex inelastic analysis, or for review using data from assumed elastic response. It has not always been used by its original name, but it or various aspects of RET have been used both directly and implicitly in much aseismic research and design. It is still suggested, as before, as a supplementary check on conventional stress or "force" analyses. There is much more to it than a means of reducing elastic spectra.

Observation of Building Damage

The careful observation and recording of building damage, and lack of damage, is an important part of earthquake engineering which must be continued. However, it is sometimes overlooked by engineers and usually by others, that observation of damage is only one aspect of 30 or more aspects in earthquake engineering. For example, detailed analysis to learn exactly why things happened, or didn't happen, is perhaps more important than the event itself. Even more important is providing reasonable and economic means to prevent such events, if serious, in the future. The observation and recording of damage is vital, but it is the tip of the iceberg -- widely seen, but not the center of gravity.

It is unfortunate that in some cases misleading or vague information has been released from the field or by the designers as to the design basis for damaged buildings, especially those in other countries. For example, some of the damaged South and Central American reinforced concrete buildings have been said to have been "designed to United States codes." Now what is a "United States code", especially many years ago when the buildings were designed? Is it the old ACI specification with small column ties far apart even near the joints; is it an UBC code -- if so, which one, etc.? Is it "ductile" concrete, and if so by what definition?

Detailed Analysis of Building Damage, or Lack of Damage

This is a vital subject and one in which there has been much accomplished. More should be done, especially on undamaged as well as damaged buildings if local ground motion records are available. Damage and lack of damage should be reconciled, preferably on several similar buildings to establish statistical parameters. The objective is to learn more, to improve analysis and design techniques, and to be able to obtain more economical as well as more reliable earthquake-resistant buildings. This work requires a combination of the most advanced technical procedures and the experienced judgment of seasoned designers.

This writer can not agree, however, that "only by exposure to real earthquakes will we be able to test our design procedures." In the first place, earthquakes are highly random, and they and their effects vary from A to Z. In the second place, a given earthquake may not really test the structure.* In the third place, we can't wait that long, nor do we have to. Good analysis and theory seasoned with experience can provide a better evaluation of most buildings than one or two earthquakes that may or may not occur in the lifetime of the structure. But this type of work has to be much more than a code-type analysis.

To analyze reinforced concrete buildings it is important to get complete drawings, or else make drawings and details from field investigations; also it is important to get test data, or else make tests of sample materials. The analysis should be done by two or more methods, where appropriate, to compare the results. Allowances should be made for period and damping variations. The conclusions should be limited to the specific data and situation at hand without extrapolations or generalizations that could be misleading.

Building Codes

The codes are improving but (a) they haven't been all that bad when good judgment has been blended with careful analyses; and (b) they will never be perfect documents because of their necessary time lag behind new knowledge, compromises, and idealizations. Special structures will still need to have more input than "passing a building code." Some damage must be expected in spite of getting a building permit under any seismic code. Few owners realize this today. Applicable codes must be met, but that alone should not be considered as a substitute for good engineering judgment and theory. However, any code exceedances need owner approval.

Statistics, Probabilistic Theory, Risk Evaluation

By whatever name, the recognition that there are random variables and that they follow certain laws and have certain parameters is a vital part of earthquake engineering. This subject has been gaining in popularity in the last 10 years or so but it has further to go. It won't solve all the problems but it "puts a handle" on many of them and it improves communication, even with the layman. It is also an aid to judgment and the handling and recording of complex data. It is a legitimate part of earthquake engineering but not a panacea as the public sometimes believes. The writer has used formal probabilistic theory, statistics and risk evaluation for over a decade on many complex problems including some from the fault as the source to the response of equipment high in a structure. He has also used it less formally over decades of consulting practice.

The conventional deterministic procedures may lead to gross conservatisms, especially where "enveloping" is used for various steps along the way [6].

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*An unwounded soldier coming out of battle is not bulletproof!

SUBJECTS APPLICABLE TO REINFORCED CONCRETE BUILDINGS

The items discussed above apply to all buildings, including reinforced concrete buildings. The subjects in this section apply only to reinforced concrete buildings.

Measurements and Tests of Full Size Buildings

There are several ways to test buildings in order to obtain dynamic properties such as periods and damping. One of the best is to measure actual motion during an earthquake. Almost as useful, and easier to get, are measured (instrumental) records of real buildings responding to man-made ground motion such as from high energy explosives or underground nuclear detonations. Much of this has been done in the last 10 years and much valuable information has been obtained about the characteristics of real buildings [13,14,27,41].

Other methods include measuring ambient motion or wind-induced motion, forced vibration, pull-and-release or bump tests, rocket-induced, and human-induced motion. Some of these techniques go back 40 to 50 years [28,29]. They have produced very useful information on periods, damping and mode shapes and variations in these with amplitude, repetitions, time and other factors. It has been found that the natural periods of some, but not all, buildings vary considerably, even without damage per se [13]. The damping may vary also, generally increasing with amplitude. Natural periods have been found to be the same, or closely similar no matter how the motion was induced, providing the amplitudes are about equal.

Two 4-story reinforced concrete frame test structures at the ERDA Nevada Test Site have provided much valuable information from many types of tests including long-continued forced vibration to extreme distortions in the damaging range [14,30]. It is expected that one of these structures, already damaged, will be repaired by epoxy or other conventional methods and then be subjected to severe motion to simulate the behavior of repaired buildings in damaging earthquakes [31].

Tests of full size buildings provide much valuable information under controlled conditions and should be continued in the future wherever feasible. The test data should be compared to code and analysis results and reconciliations made where indicated.

Large Scale Tests of Members, Assemblies, Elements and Joints

The testing of full size or large scale members, assemblies, elements, and joints under static, reversed, cycling, and dynamic forces with careful measurements in controlled conditions such as by Bertero, Popov, and others [10] is very valuable work. The value lies in determining the real properties of complex reinforced concrete members and joints over the entire range from small strains to ultimate loads. From this it is possible to improve the design procedures for better performance of the buildings with increased confidence and at optimum cost. It is especially important that the capacity to do work (reserve energy) be measured, and increased, so that maximum ductility can be obtained and the energy demands of severe earthquakes can be resisted without failure or excessive deterioration under reversals and cycling.

Damping determinations are needed under various strain levels or ductilities, loading rates, histories of loading, deterioration stages, etc. Variations in natural frequencies or stiffnesses are also needed under similar conditions.

Shaking Table Experiments

Shaking table experiments with reinforced concrete models or with full size elements such as being conducted by Clough, Penzien, and others [10] are most desirable, especially where various ground motion records are used to shake the test specimen well into the inelastic range. The data obtained on response, period variations, damping, modal combinations, damage and lack of damage, and with repetitions where possible under different disturbances are valuable and can not be obtained as well, if at all, by other procedures. More should be done on combined soil-structure systems in the 2D, 3D and inelastic regimes.

Layout of Buildings

The manner in which a building is planned geometrically is usually vital to its earthquake integrity, especially to its economy of earthquake resistance. Experienced designers of buildings know that of two buildings passing a given earthquake code, one may be much better than the other because of its layout. The buildings may be of the same height and plan dimensions. Too many designers fail to recognize that the real forces, the real strains, the real conditions, may be much more severe than the code-required forces and shears would indicate. They should be thinking of the inelastic behavior of the building as they design it to code elastic stress levels. Unfortunately, few structural engineers have much control over the layout of buildings. This is usually done by the architect and perhaps the owner.

A recent well illustrated paper [32] discusses the importance of layout. It has been discussed in prior years as well and it is well known to experienced earthquake engineers.*

The more changes, especially significant or abrupt ones in plan or in elevation, the more potential problems. Also symmetry of rigidity about the two horizontal axes is most desirable (even a symmetric structure will be subject to some torsion in the elastic range, and possibly a great deal of torsion in the inelastic range).

However, it is possible to have both symmetry of rigidity about the two horizontal axes and a perfectly rectangular building in plan and elevation and still have a less-than-ideal layout. Reference is made to

*Another case where more formality may be indicated.

a "core" building where most if not all the resistance is confined to inner walls, generally surrounding the stairs, elevators and utility shafts. The buildings look nice, they may have that graceful slender-column exterior appearance at the first story, they may pass a building code, but they may also have a very low polar moment of inertia of rigidity about the vertical axis. The result in a major event could be a severe twisting that could do a lot of damage and be dangerous. It is almost a certainty that buildings that go into the inelastic range (and most will) will do so in an asymmetric manner no matter how symmetric they may be in the elastic range. Research should be done on core buildings; in the meantime they should get special attention.

Another potential (and historical) problem is corner columns. There is indication that with 2D and 3D action, with or without "accidental" or inelastic torsion, corner columns designed by most codes could have trouble [33]. Not only are certain geometric problems and dynamic problems possible, but corner column joints are not generally as well confined as other joints, and there may be less compressive stress to aid in shear and flexural resistance.

Types of Concrete Buildings

Buildings come in various shapes, sizes, and types. There are "box" buildings with nothing but walls, "framed" buildings, open column buildings with slab floors and no beams, frames or slab buildings with "shear" walls, framed buildings of various types with full or partial filler walls of either strong or weak materials, braced buildings, open buildings, etc. And, of course, there are cast-in-place, precast, prestressed, poststressed, continuous pour, upside-down construction, and all sorts of combinations. Cost is the problem. Earthquake resistance is also a problem.

If the real forces were the same as the code forces and if all code requirements were met, there would be no serious problem, with one or two exceptions. Those exceptions have to do with the injection of partial-height rigid walls between columns. The partial height walls may be "filler" and not be subject to control in most codes, but they can "punch" the columns into local shear failures. Full filler walls are not as bad but may also cause trouble.

But real earthquake forces and shears may be several times code-required forces and shears. Although there are many factors that may reconcile this difference [6], they may in some cases fail to do so. In such situations certain types of buildings may be better than others. One key consideration might be to use that type whose rigid base period is most removed from any dominant site periods. Frame action alone may be desirable, but it may allow so much story distortion as to cause excessive "non-structural" damage. Shear walls can be used, often to advantage, but they shouldn't just be dropped in here and there -- they should be considered part of the structural system. Shear walls, on the other hand, may increase rigidity and response acceleration considerably over that of a frame. They also decrease any uplift attenuation. These matters, and others, all need to be considered in research and in design.

Long, narrow, box-type buildings without intermediate transverse shear walls tend to vibrate in a mode in which the floor diaphragm predominates [34]. This in turn tends to twist the end shear walls and add torsional shear to other shear. The Arvin High School walls no doubt felt this action in 1952 [35].

Many buildings have such rigid vertical elements as compared to the horizontal elements that the buildings are essentially vertical cantilevers [36]. This results in coupled wall problems (in spandrels) that may or may not be serious, cantilever-type period ratios, and rather short fundamental periods for the height. Various types of buildings with reference to the ratio of vertical to horizontal stiffness have been categorized [36].

<u>The Flexible First Story and Other Attenuators</u>

The flexible first story resurfaces about every 10 or 15 years [37]. It was a popular subject 40 and 50 years ago [38,39,40] and a few buildings were erected with this concept in mind. Actually, a great many buildings have relatively flexible first stories because of their extra height and many openings for doors and windows. There is little doubt that a "soft" or flexible first story tends to reduce the lateral forces in the structure [38]. A lifting or tilting foundation tends to do the same.

The problem in the real world is that the actual distortions from ground motion may grossly exceed those derived using code lateral forces, and secondary stresses ($P\Delta$, etc.) may become serious. There is also the torsional problem to be considered whether or not the building code so specifies.

A building with a very flexible first story as compared to the other stories tends to act as a one-mass system if the soil is rigid. Its period of vibration will vary from that of a more conventional building of the same mass and geometric proportions. This may be good or bad, depending upon the ground motion. Its total energy absorption may be less than that of a more conventional building.

In general, flexible first story buildings should be approached very carefully and have complete dynamic analysis with a range of time histories and response spectra of realistic ground motion.

The same can be said for most of the other attenuators and energy absorbing devices. They have theoretical merit but should be used only with adequate research and caution unless one knows just how far the structure will move, and what will stop it, and with what reserve value, and what will keep everything in line, in place, and functional at all times.

Ductile Concrete (DC)

The writer wishes to clarify that "ductile concrete" [8,9] as he defines it is not just the use of special transverse reinforcement in beams, girders, columns, and joints, but also the more general aspect of designing so that: shear failure can not occur before flexural stretching of longitudinal bars; flexural stretching of longitudinal bars will precede and thus prevent flexural compression failure in concrete; and confining compression areas so that if concrete should fail locally in compression it would be adequately confined and develop the most ductility. The minimum amount and type of transverse reinforcement to do all this may still be somewhat conservatively defined.

A challenge is to develop the most reliable reinforced concrete column with the optimum combination and quantity of steel and concrete for the maximum confinement and ductility in interaction.

Real versus Specified Concrete Strength

The specifications call for 28-day strength, f'c. They also don't allow over 5% or 10% of the test cylinders to fall below f'c. The contractor may go to greater f'c "to be sure". The result is that the average f'c on the job is much greater than that specified or used in the design calculations. Then the earthquake is delayed from 28 days to maybe 28 years, and f'c is considerably stronger, especially with certain types of cement. The overall result is that the concrete is much stronger than given credit for and it is also more rigid.

A further consideration is that laboratory test data are plotted for parameters or for dimensionless ratios. Then under the usual deterministic procedures someone, or a committee, draws a curve that falls below 95% to 100% of all the test points, for "safety". This plot may relate shear to f'c, for example. The real average shear relative to f'c at 28 days may be 15% to 30% greater than the drawn line or curve would indicate.

Combining these conservatisms -- the tests for shear, the concrete mix, the age increase, perhaps the contractor's "safety" allowance as well, we find the true shear value to be much greater than allowed for -- maybe 50% to 100%. The rigidity (E_c) will also be greater; this may be good or bad depending upon the spectral demands of the earthquake. Certainly, the extra strength is good.

It is suggested that this always be considered in research and be a bonus in design for the case where the real forces exceed code forces. However, when we resort to probabilistic designs, as we should, such matters can be handled better by using real mean values and dispersions from the mean values together with realistic earthquake demands [6].

Material Research

More should be done on the control and properties of rebars. Also the trend toward high stress should be watched to insure that elongation and ductility are not sacrificed. Very "brittle" steel should be avoided -- even in heavy columns. Bar splices, welded and mechanical, have to be watched. Bundled bars must be compatible with ductile design and be adequately tested.

Concrete strength and type should be studied as trends change, such as to higher f'c's.

Stress-strain laws should be reinvestigated where strengths have increased above those for prior test levels.

Can fibers be used with safety?

Bond-slippage relationships under cycling and inelastic conditions should be well known, controlled, and possibly improved.

Confined and unconfined concrete under different states of shear and strain and under 1D, 2D, and 3D action should be studied more [42].

CONCLUSION

Under today's conditions, including the partial replacement of judgment with computer output, it is necessary to conduct research in more depth than before, and to formalize procedures and documentation even in areas about which there may be a fairly sound base. There is also an added value in this process of education and technology transfer.

A great deal has been learned about how better to design and construct reinforced concrete buildings to resist severe earthquake motion more reliably than in prior decades. A problem is how to make them more economical; money wasted in the initial construction is gone forever. Another problem, for all types of buildings, is the risk level for those constructed long ago without the benefits of current knowledge -- some of those buildings may be good ones, for various reasons including the judgment of the designers, but many are not. This is a difficult social problem. There are, indeed, many remaining and some new problems but the road ahead looks clear to even better buildings and increased public safety.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED
CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

AN OVERVIEW OF THE STATE-OF-THE-ART IN EARTHQUAKE RESISTANT CONCRETE BUILDING CONSTRUCTION IN CANADA

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INTRODUCTION

The purpose of this report is to provide to the participants of the workshop an overview of the state-of-the-art in Earthquake Resistant Concrete Building Construction (ERCBC) in Canada.

The close technical co-operation that exists between U.S. and Canadian engineers means that many aspects of the Canadian state-of-the-art in ERCBC are essentially identical to those of the U.S. Hence rather than duplicating material that will presumably be covered by the U.S. reporters, we will restrict our attention primarily to those aspects which are specifically Canadian.

After briefly reviewing the need for earthquake resistant construction in Canada we will summarize the development of the Canadian code provisions for seismic design "loads". Canadian code provisions for detailing earthquake resistant concrete buildings and the manner in which some of these provisions are applied in practice will then be described. A brief listing of current Canadian research in ERCBC plus our opinion of the specific problems that still need to be solved will conclude the report.

EARTHQUAKE RISK IN CANADA

The Division of Seismology and Geotechnical Studies of the Earth Physics Branch of the Department of Energy, Mines and Resources (EMR) of the Federal Government of Canada is responsible for monitoring seismic ground motion for seismological data management and for seismological studies of earth dynamics and earth structures to insure the availability of information and expertise for industry, the public and the government.

The following brief discussion of the earthquake risk in Canada is based on a 1975 EMR report by Whitham and Hasegawa [1].

The current instrumentation network in Canada is capable of recording all earthquakes with magnitudes of 4 or more, and in certain areas there is a 90% probability of locating earthquakes down to a magnitude of 3 or even less. Some 200 to 300 earthquakes occurring in Canada are recorded each year. On the average, 14% of the earthquakes are located in Eastern Canada, 27% in Western Canada and 59% in the North, with very few being located in Central Canada.

In the last 75 years, 7 earthquakes, with magnitudes of 7 or greater, occurred in Canada, as listed below:

Year	Location	Magnitude
1918	Vancouver Island near Estevan (West)	7
1925	St. Lawrence River near the mouth of the Saguenay (East)	7
1929	Grand Banks, Newfoundland (East)	7.2
1929	Queen Charlotte Sound (West)	7
1933	Baffin Bay (Arctic)	7.3
1946	Strait of Georgia between Powell River and Courtenay, B.C. (West)	7.3
1949	Northern tip of Queen Charlotte Islands (West)	8.0

In the last fifty years, there has been on the average, one earthquake each decade with a magnitude greater than 6 in Eastern Canada, and two each decade with magnitudes greater than 6.5 in Western Canada. Major damage in the East was also reported from the Cornwall earthquake of 1944, which had a magnitude of 5.9. When it is recalled that the San Fernando Earthquake of 1971 (magnitude 6.4) caused 58 deaths and over half a billion dollars of damage, the necessity of establishing earthquake-resistant design regulations in Canada becomes evident. The mechanism used for this purpose is the National Building Code of Canada (NBC).

NATIONAL BUILDING CODE OF CANADA (NBC)

In Canada, which is made up of 10 provinces, 2 Districts and the Federal Government, the responsibility for regulating building safety belongs solely to the Provinces. Provincial statutes, in turn, place the responsibility of regulating building safety on to the municipalities. In the past, each municipality was free to establish its own standards (or codes) and enforce them as it saw fit. The maze of by-laws that at one time existed across Canada was labelled as being probably "the greatest obstacle to progress in building" [2]. To overcome this obstacle, work on a National Building Code was started in 1937 and the first code was published in 1941 by the National Research Council of Canada.

Although the National Building Code is written in code language, it has no official status until adopted as a by-law by a municipality, and in the past, the availability of an advisory document did not necessarily reduce the maze of by-laws and regulations.

Recently, the final step to remove the confusion was taken by some of the Provinces. For example, in 1974 the Province of Ontario passed a new Building Code Act, which made the Ontario Building Code (OBC) [3] a regulation under this Act. Thus OBC applies throughout the Province, is legally binding, and uses the National Building Code of Canada as its basis for structural regulations.

The National Building Code of Canada is published by the National Research Council (NRC) of Canada through its Associate Committee on the National Building Code (ACNBC). This committee consists of 27 individuals with 10 members from Government, 16 from Industry, and 1 from a University, all appointed by the National Research Council of Canada.

The responsibility for the Structural part of the NBC rests with the Standing Committee on Structural Design. This Standing Committee has 22 members, with 5 members from Government, 11 from Industry, and 6 from the Universities.

So far as ERCBC is concerned, there are two other national committees that are of interest. The Canadian National Committee on Earthquake Engineering (CANCEE) advises the Standing Committee on Structural Design on all matters related to earthquake resistant design. There are at present 22 members of CANCEE, with 6 members from the Government, 7 from Industry, and 9 from the Universities. The CSA/NBC Joint Committee on Reinforced Concrete Design is a joint Canadian Standards Association (CSA) and National Building Code Committee, responsible for the structural reinforced and prestressed concrete code provisions. The recommendations of this committee, after approval by the CSA, becomes a Canadian Standard. The current Standard (CSA A23.3-1973), through reference to it in the NBC, is part of the NBC of Canada. This Standard is similar to the American Concrete Institute Code (ACI 318-71). The present membership of this committee numbers 14, with 3 members from the Government, 7 from Industry, and 4 from the Universities.

The Standards and Codes formulated by the above committees are circulated at their final draft stage for public comment. Nevertheless, the recommendations are not published in the literature and hence are not subject to wide public discussion prior to their publication in final form. This we believe is the single most serious shortcoming of the present Canadian system.

DEVELOPMENT OF NBC OF CANADA SEISMIC DESIGN PROVISIONS

In order to place into perspective the current seismic provisions of the National Building Code, we will trace their evolution by briefly describing the seismic provisions of successive versions of the NBC.

NBC (1941)

The seismic provisions of the first National Building Code of Canada were based on the concepts presented in the 1937 Uniform Building Code (UBC). The lateral earthquake force was assumed to act at the centre of gravity of the structure and to have a magnitude given by a seismic coefficient, which depended on the bearing capacity of the soil, times a building weight, which was taken as the dead load plus one half of the live load.

NBC (1953)

On the basis of earthquake zoning established by the U.S. Coast and Geodetic Surveys, the second edition of NBC (1953) introduced four earthquake probability zones. The resulting zoning map (see Figure 1), which remained in force until 1970, had two main drawbacks: discontinuities existed at some

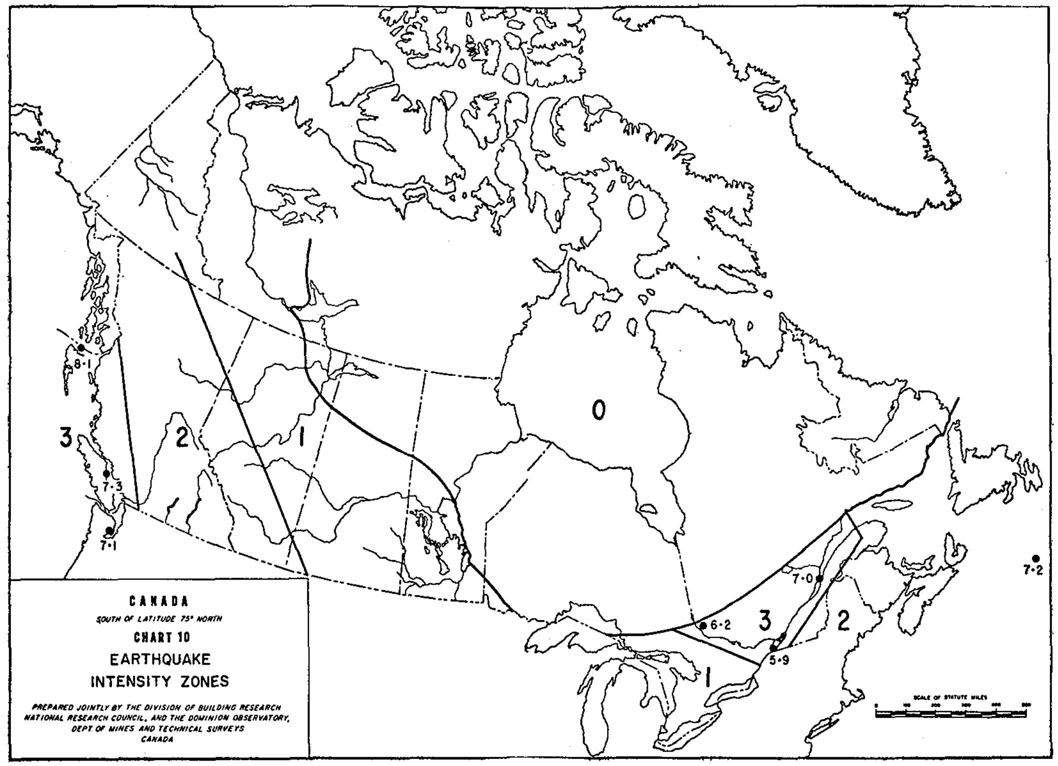

FIGURE 1: SEISMIC ZONING MAP OF CANADA (1953-1970)

zone boundaries (e.g. zone 0 lay next to zone 3 in some locations); and unrealistically high ratings of risk were assigned to some cities (e.g. Ottawa and Montreal).

The seismic design provisions of the 1953 NBC were primarily based on the 1949 edition of the UBC. The ratio of the seismic base shear to the building weight (taken as the dead load plus 25% of the design snow load) was assumed to be a function of the building height, but to be independent of soil conditions. Figure 2, which will be used to illustrate the evolution of the Canadian seismic loading provisions, shows the practical range of values for this 1953 base shear coefficient.

NBC (1960)

The provisions for seismic design of the third edition of NBC (1960) remained essentially the same as those of the 1953 edition.

NBC (1965)

In the 1965 Code the seismic base shear coefficient was assumed to be a function of: (1) the seismic risk zone, (2) the type of construction, (3) the importance of the building, (4) the soil conditions, and (5) the number of storeys of the building.

Figure 2 gives the values of the base shear coefficient obtained from the 1965 Code for a ductile moment resisting frame building in zone 3. It can be seen that the introduction of a factor recognizing the importance of ductility resulted in a significant lowering of the seismic design base shear for certain types of structures.

While the 1965 NBC adopted the 1955 UBC expression relating base shear to the number of storeys, it required that this base shear be distributed over the height of the building in a manner derived from the 1961 edition of UBC.

The 1965 edition was the first NBC Code to include torsional considerations in the seismic provisions. These torsional clauses were based on the then existing Mexican Code and still remain in the 1975 NBC Code.

As an alternative to a simple static analysis the 1965 NBC code permitted the design loading due to earthquake motion to be determined by a dynamic analysis "where such an analysis is carried out by a person competent in this field of work."

NBC (1970) [4]

Based on the work of Milne and Davenport [5] the Canadian seismic zoning map was completely revised for the fifth edition of the NBC (1970). The 1970 zoning map (see Figure 3) was derived by calculating and contouring the peak horizontal firm ground acceleration amplitudes that have a probability of exceedance of 0.01 in one year (A_{100}). On the basis of work by Ferahian [6] it was stated [7] that for typical cities in zone 3 using the 100 year return period for seismic forces would mean a comparable probability of collapse to that resulting from the 30 year return period for wind loads used by NBC.

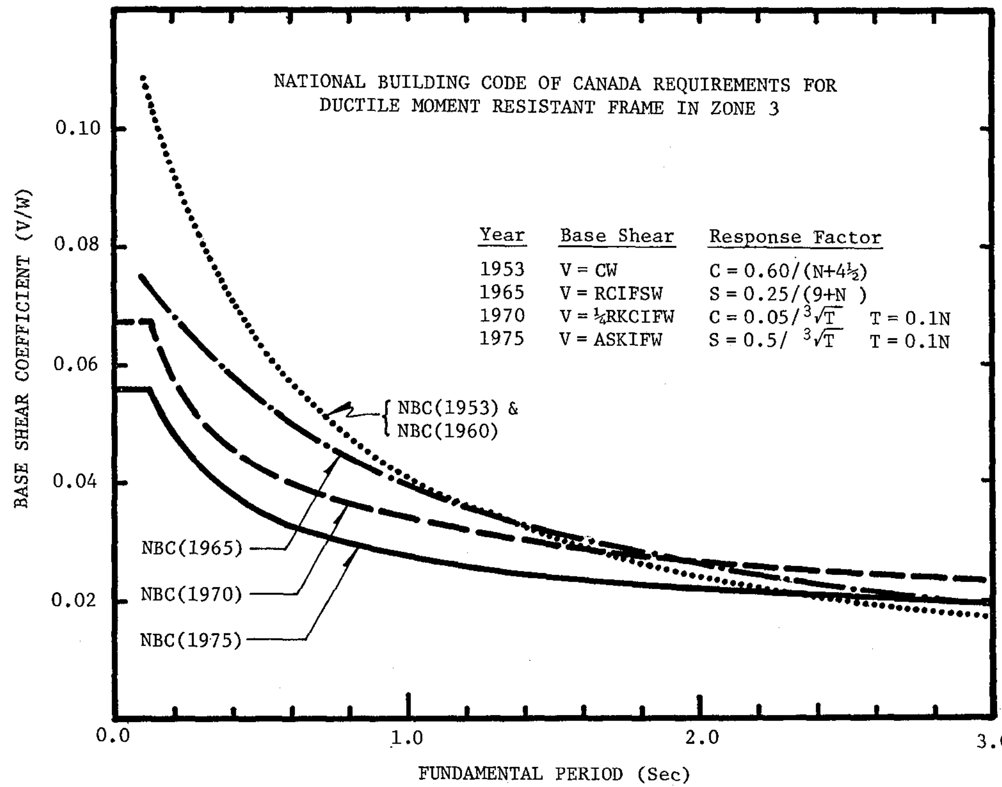

FIGURE 2: VARIATION OF NBC BASE SHEAR COEFFICIENTS WITH TIME

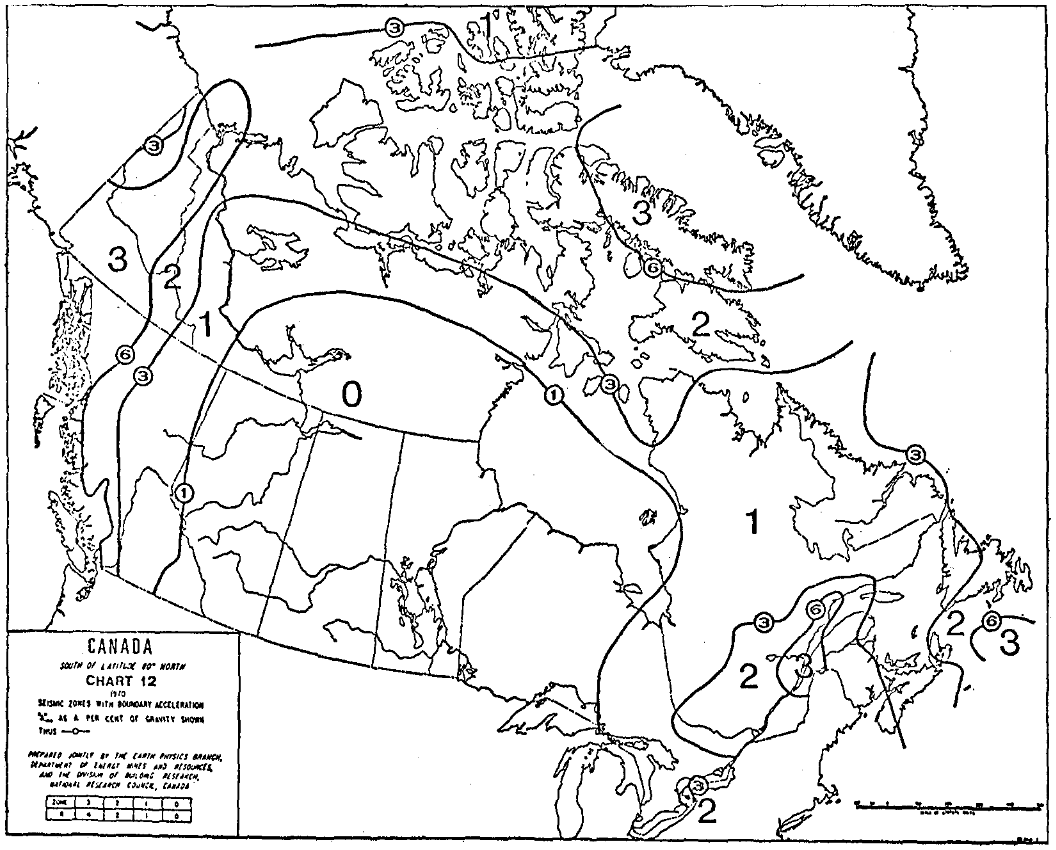

FIGURE 3: SEISMIC ZONING MAP OF CANADA (1970)

The probability of earthquake occurrence for a city on the Canadian West Coast and in an area north of Quebec City was found to be generally comparable to the probabilities for a city of similar size in California [7].

Seismic zone and seismic zone factors (R) to be used in computing the base shear were selected as shown in Table I.

TABLE I - SEISMIC ZONES NBC (1970)

Zone	R	Range of A_{100} (% of gravity)	Zone Description
0	0	$1 > A_{100}$	No damage
1	1	$3 > A_{100} \geq 1$	Minor damage
2	2	$6 > A_{100} \geq 3$	Moderate damage
3	4	$A_{100} \geq 6$	Major damage

It is important to note that, in NBC (1970), although the acceleration with a 100 year return period (A_{100}) was used to draw the boundaries of the seismic zones, the actual value of A_{100} did not enter into the seismic design force calculations. The minimum lateral seismic force assumed to act non-concurrently in any direction was given as:

$$V = \frac{R}{4} KCIFW \qquad \ldots\ldots (1)$$

where R = Seismic zone factor (0, 1, 2 or 4) as defined in Table I

K = Numerical coefficient, depending on construction type, 6 possible types as in UBC (1961)

$C = 0.05/\sqrt[3]{T} \ngtr 0.10$

T = Fundamental period, taken as $0.05\, h_n/\sqrt{D}$ or as 0.1N.

I = Importance factor (1.0 or 1.3)

F = Foundation factor (1.5 or 1.0)

The range of seismic base shear coefficients which result from the application of the 1970 NBC rules for ductile frame buildings in zone 3 is shown in Figure 2. It can be seen that for buildings with fundamental periods less than about 1.8 seconds the new 1970 provisions once more resulted in a substantial lowering of the lateral seismic design forces.

In an attempt to account for higher mode effects the 1970 code required a portion of the total lateral load to be applied as a concentrated load at the top of the structure. For the same reason a reduction in the calculated overturning moment was permitted.

NBC (1975) [8] - Static Load Procedure

As this is the current version of the National Building Code, its seismic provisions will be described in somewhat more detail.

TABLE II - VALUES OF THE COEFFICIENT K AS GIVEN BY THE 1975 CANADIAN CODE [8]

Case(1)	Type or Arrangement of Resisting Elements	Value of K
1	*Buildings* with a ductile moment-resisting space frame(2),(3) with the capacity to resist the total required force.	0.7
2	*Buildings* with a dual structural system consisting of a complete ductile moment-resisting space frame and ductile flexural walls(4) designed in accordance with the following criteria: The frames and ductile flexural walls shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the flexural walls and frames. In this analysis the maximum shear in the frame must be at least 25 per cent of the total base shear.	0.7
3	*Buildings* with a dual structural system consisting of a complete ductile moment-resisting space frame and shear walls(5) or steel bracing designed in accordance with the following criteria: (a) The shear walls or steel bracing acting independently of the ductile moment-resisting space frame shall resist the total required lateral force. (b) The ductile moment-resisting space frame shall have the capacity to resist not less than 25 per cent of the required lateral force, but in no case shall the ductile moment-resisting space frame have a lower capacity than that required in accordance with the relative rigidities.	0.8
4	*Buildings* with ductile flexural walls(4) and *buildings* with ductile framing systems not otherwise classified in this Table as Cases 1, 2, 3 or 5.	1.0
5	*Buildings* with a dual structural system consisting of a complete ductile moment-resisting space frame with masonry infilling designed in accordance with the following criteria: (a) The wall system comprising the infilling and the confining elements acting independently of the ductile moment-resisting space frame shall resist the total required lateral force. (b) The ductile moment-resisting space frame shall have the capacity to resist not less than 25 per cent of the required lateral force.	1.3(6)
6	*Buildings* (other than Cases 1, 2, 3, 4 and 5) of (a) continuously reinforced concrete, (b) structural steel, and (c) reinforced masonry shear walls.	1.3
7	*Buildings* of unreinforced masonry and all other structural systems, except Cases 1 to 6 inclusive.	2.0
8	Elevated tanks plus full contents, on 4 or more cross-braced legs and not supported by a *building*, designed in accordance with the following criteria: (a) The minimum and maximum value of the product SKI shall be taken as 1.2 and 2.5 respectively. (b) For overturning, the factor J as set forth in Sentence 4.1.9.1.(14) shall be 1.0. (c) The torsional requirements of Sentence 4.1.9.1.(15) shall apply.	3.0

(1) Explanatory notes on the various cases can be found in the Commentary on Effects of Earthquakes in NBC Supplement No. 4, "Commentaries on Part 4 1975."
(2) A space frame is a 3 dimensional structural system composed of interconnected members laterally supported so as to function as a complete self-contained unit with or without horizontal diaphragms.
(3) A ductile moment-resisting space frame is a space frame that is designed to resist the specified seismic forces and that, in addition, has adequate ductility or energy-absorptive capacity.
(Information on ductile moment-resisting space frames can be found in the Commentary on Effects of Earthquakes in NBC Supplement No. 4, "Commentaries on Part 4 1975.")
(4) A ductile flexural wall is a ductile flexural member cantilevering from the *foundation* consisting of a ductile reinforced concrete wall designed and detailed according to CSA A23.3-1973, "Code for the Design of Concrete Structures for Buildings," Chapter 19, Special Provisions for Seismic Design.
(5) Shear walls may be either flexural walls or shear walls as defined in CSA A23.3-1973, "Code for the Design of Concrete Structures for Buildings," Chapter 19, Special Provisions for Seismic Design.
(6) Except as required by Sentence 4.1.9.3.(1).

Minimum lateral seismic force--In the 1975 Code the minimum lateral seismic force, V, is specified as:

$$V = A.S.K.I.F.W. \qquad \ldots\ldots (2)$$

where the terms of this expression are defined as follows:

A is the assigned horizontal design ground acceleration for the zone in question. The values of A, which correspond to the A_{100} values discussed with respect to the 1970 Code, are 0.00, 0.02, 0.04 and 0.08 for seismic zones 0, 1, 2 and 3 respectively.

S is a seismic response factor taken as $0.5/\sqrt[3]{T}$ but need not be taken as greater than 1.00.

K is a numerical coefficient reflecting the influence of the type of construction on the damping, ductility and/or energy-absorption capacity of the structure. The numerical values of K for the various types of construction considered are given in Table II.

It can be seen that while the basic concept of the coefficient K is the same as that used in the 1976 UBC [9] the classifications and the numerical values are significantly different. The implications of this table and the explanatory footnotes that accompany it on the earthquake resistant design of Canadian reinforced concrete buildings will be discussed later in this report.

I is an importance factor taken as 1.3 for schools and for buildings designed for post-disaster services and as 1.0 for all other buildings.

F is a foundation factor accounting for the influence of the soil conditions. The values of F are given in Table III.

TABLE III - VALUES OF THE FACTOR F [8]

Type and Depth of *Soil*[1]	F
Rock, dense and very dense coarse-grained *soils*, very stiff and hard fine-grained *soils*; compact coarse-grained *soils* and firm and stiff fine-grained *soils* from 0 to 50 ft deep	1.0
Compact coarse-grained *soils*, firm and stiff fine-grained *soils* with a depth greater than 50 ft; very loose and loose coarse-grained *soils* and very soft and soft fine-grained *soils* from 0 to 50 ft deep	1.3
Very loose and loose coarse-grained *soils*, and very soft and soft fine-grained *soils* with depths greater than 50 ft	1.5[2]

W is the dead load of the building plus 25% of the design snow load.

The base shears that result from the application of Equation 2 to ductile moment resisting frame buildings in zone 3 are shown in Figure 2. It can be seen that the 1975 provisions result in base shears 20% less than those given by the 1970 Code.

The 20% reduction in base shear resulted from a recommendation by CANCEE to so reduce the earthquake loads. In fact, the seismic response factor S was derived so that the term A.S in the 1975 Code would have a value 20% less than the value of the term RC/4 from the 1970 Code. It is important to realize that this procedure for determining S means that although the 1975 Code refers to a "design ground acceleration" the Code does not necessarily require a building to be designed for a peak horizontal ground acceleration corresponding to 1 in 100 probability of annual exceedance. The choice of the factor S related the earthquake risk level to that used in the 1970 Code rather than directly to the acceleration values.

Distribution of the lateral seismic force--The 1975 Code retained the 1970 requirements for the distribution of the lateral seismic force. These requirements are essentially the same as those employed in the 1961 edition of UBC.

Overturning moment reduction coefficient--The concept of an overturning moment reduction coefficient, J, was retained from the 1970 Code but the values of this coefficient were made closer to 1.0; whereas in 1970 the coefficient reduced to about 0.6 for buildings with periods greater than 3 seconds, in 1975 the values for J were set as:

$$J = 1.0 \quad \text{if } T \leq 0.5 \text{ sec.}$$
$$J = 1.1 - 0.2T \quad \text{if } 0.5 \text{ sec} \leq T \leq 1.5 \text{ sec}$$
$$J = 0.8 \quad \text{if } T \geq 1.5 \text{ sec.}$$

Torsional moments--The 1975 Code extended somewhat the torsional requirements of the 1970 Code by requiring that the design eccentricity at each floor be computed by whichever of the following two equations produces the greater effects:

$$e_x = 1.5e + 0.05\ D_n \quad \text{or}$$
$$e_x = 0.5e - 0.05\ D_n$$

where e is the computed eccentricity between the centre of mass and centre of rigidity and D_n is the plan dimension of the building in the direction of the computed eccentricity.

In the event that the maximum design eccentricity, e_x, exceeds 0.25 D_n, the Code requires either that a dynamic analysis be performed or that the computed adverse effects of torsion be doubled.

Drift limits--The NBC Code states that in order to obtain realistic values of anticipated lateral deflections of a storey relative to its adjacent storeys, the values obtained from an elastic analysis should be multiplied by 3. The Commentary [10] on the Code recommends an inter-storey drift limit of 0.005 times the storey height.

NBC (1975) - Dynamic Analysis Approach

In lieu of the design procedure described above the 1975 edition of NBC permits the design earthquake loading to be determined from a dynamic analysis. The details of this approach are given in the Commentary [10] to the Code.

The Commentary makes clear that the major use of the dynamic approach is expected to be in the design of "unusual or complex structural configurations for which the static NBC procedures are necessarily crude." It is stated that "for regular buildings, the static NBC requirements and the recommended dynamic procedure should give similar results." It will be demonstrated later in this report, that this statement is of doubtful validity.

Design ground motions--The characteristics of the design ground motions are defined in terms of peak ground motion bounds which in turn are linked to the seismic risk level by way of the peak horizontal ground acceleration corresponding to 1 in 100 probability of annual exceedance (i.e. A_{100}). For a peak ground acceleration of 1.0 g the corresponding velocity bound is 40 in./sec. (1016 mm/sec.), and the displacement bound is 32 in. (813 mm). For other values of peak ground acceleration the bounds are scaled linearly.

Design elastic response spectrum--The design average elastic response spectrum is obtained by multiplying the peak ground motion bounds by multipliers which depend on the assumed value of damping. For reinforced and prestressed concrete the suggested design value of damping is 5% of critical. For this value of damping the multipliers for the acceleration, velocity and displacement bounds are 3.0, 2.0 and 2.0 respectively. It is stated that these amplification factors were adapted from Newmark, Blume and Kapur [11] and Newmark and Hall [12].

Design inelastic response spectrum--To account for inelastic deformation the average elastic response spectrum is modified by terms depending on the structural system ductility factor μ. The elastic spectral acceleration is divided by μ for modal periods falling in the range of velocity and displacement bounds and by $\sqrt{2\mu-1}$ for modal periods along the acceleration bound. The values of μ for various building types are given in Table IV below.

TABLE IV - VALUES OF STRUCTURAL DUCTILITY FACTORS [10]

Building Type	Structural Ductility Factor
Ductile moment resisting space frame	4
Combined system of 25 per cent ductile moment resisting space frame and ductile flexural walls	3
Ductile reinforced concrete flexural walls	3
Regular reinforced concrete structures, cross-braced frame structures and reinforced masonry	2
Structures having no ductility, plain masonry	1

Foundation factors and importance factors--Unless a more detailed analysis of the influence of soil conditions is carried out considering the propagation of seismic wave from rock to surface, the average response spectrum must be multiplied by the foundation factors (Table III).

In the absence of a more detailed procedure of adjusting the probability of exceedance of peak ground acceleration and modifying the acceptable degree of plastic deformation, the Importance factors of the static design procedure are to be used as minimum multipliers to the average response spectrum.

Design forces--The design forces and interstorey drift are obtained by taking the square root of the sum of the squares of the effect from each mode. A further requirement is that the design forces should not be less than the absolute sum of the effect from any two modes. This last requirement will be dropped from the forthcoming 1977 edition of the Code.

COMPARISON OF NBC (1975) STATIC, NBC (1975) DYNAMIC AND UBC (1976) SEISMIC BASE SHEARS

The base shear coefficients obtained from the NBC (1975) static procedure, the NBC (1975) dynamic procedure and the UBC (1976) procedure for a ductile moment resisting frame building located in zone 3 are compared in Figure 4.

It can be seen that for buildings with fundamental periods less than about 2 seconds, the NBC static procedure results in seismic base shears smaller than those obtained from the UBC. Before any conclusions can be drawn from this comparison it is necessary to know the relative magnitudes of the load factors used in the two codes.

The National Building Code (1975) includes load factors, load combination factors and importance factors as part of the limit states design procedure. However, the present Code for the Design of Concrete Structures for Buildings, CSA A23.3-1973 [13], was not formulated with these new load factors in mind. (Work is currently underway [14] to examine the material performance factors needed to utilize the limit states load factors.) At present load factors for reinforced concrete structures are given in CSA A23.3 (they are very similar to the ACI 318-71 [15] values) and the load combination factors are given in the NBC Code. Thus for the earthquake resistant design of Canadian reinforced concrete structures the combined load effects that must be considered are 1.4D + 1.8E, 0.75 (1.4D + 1.7L + 1.8E) and 0.9D + 1.4E.

For the seismic design of a reinforced concrete ductile moment resisting space frame located in zone 3, the load combinations specified in the UBC are 1.4 (D + L + E) or 0.9D + 1.4E.

Thus both the design seismic base shear and the load factors can be lower for a building on the Canadian side of the border than for a comparable building subjected to a presumably comparable seismic risk on the U.S. side of the border.

Another possible source of concern regarding the Canadian seismic design provisions is illustrated in Figure 4. It can be deduced from this figure that according to the 1975 Code, the probability of a reinforced concrete building at a particular site in Canada being severely damaged or destroyed by an earthquake depends to some extent on whether it was designed by the static or the dynamic procedure. A relatively tall building analyzed by the dynamic procedure can be designed to resist less than one half of the seismic base shear required for an identical building designed by the static approach.

It is difficult to prove which is the more appropriate level of earthquake resistance, that corresponding to the dynamic approach or that

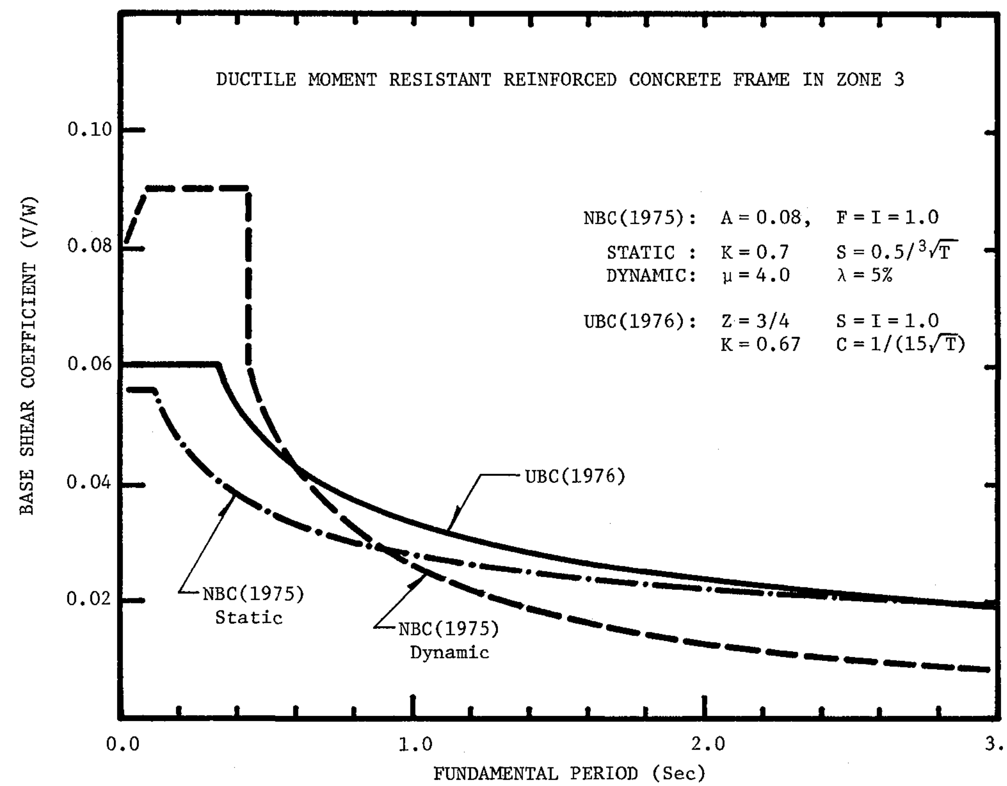

FIGURE 4: COMPARISON OF BASE SHEAR COEFFICIENTS OF NBC(1975) and UBC(1976)

corresponding to the static approach. What is certain is that the static approach gives results close to the "traditional" level of earthquake resistance, (It will be recalled that the S factor in the 1975 static expression was arranged to give a base shear 80% of that given by the 1970 expressions), while the dynamic approach relates the earthquake resistance to the peak horizontal ground acceleration with a 1 in 100 probability of annual exceedance (A_{100}).

While it may be reasonable to use the A_{100} values to define seismic risk zones throughout Canada it does not necessarily follow that these values should be actually used to calculate the seismic design forces. The 1 in 100 probability of annual exceedance means that a structure with a 50 year life will have about a 40% chance of being subjected to ground accelerations in excess of A_{100} values. Further, the relatively small data base used in the derivation of the A_{100} values and the standard errors involved in computation means that they carry a composite uncertainty up to a factor of about two [16]. Apart from questions regarding the appropriate values for design ground accelerations it has been argued that the present procedure places far too much emphasis on peak ground acceleration values to the detriment of other important ground motion characteristics [1].

Even if it was agreed that it was appropriate to design on the basis of the A_{100} values it must be recognized that the dynamic analysis procedures then employed to arrive at the design forces are not exact scientific procedures yielding exact results. Procedures such as that used to modify the elastic response spectrum to allow for the effects of ductility are far from exact when applied to structures as complex as a high-rise reinforced concrete building.

The fact that for regular buildings the dynamic procedure did not give the "similar results" envisaged by the Commentary was a cause for concern. Some of the implications of the dynamic procedure were made apparent in a recent paper by Tso and Bergman [17] and in the resulting discussion by Otani and Uzumeri [18].

Based on the concept that perhaps it was not wise to stray too far from the traditional levels of earthquake resistance, the soon to be published 1977 edition of NBC will require that the seismic base shear used in a dynamic analysis procedure not be less than 90% of the base shear obtained from the equivalent static force procedures of the Code.

SEISMIC DESIGN PROVISIONS OF THE CANADIAN CONCRETE CODE

The current Canadian code regulations for detailing earthquake resistant reinforced concrete buildings are given in Chapter 19 of the Code for the Design of Concrete Structures for Buildings, CSA A23.3-1973 [13] and in the 1977 Supplement [19] of this Code.

The first clause of Chapter 19 states that the provisions of the chapter "apply to reinforced concrete structures where required or permitted to be designed to resist earthquake forces in a ductile manner." A footnote to this clause informs us that "Seismic coefficients for buildings designed to have ductile moment resistant space frames and/or ductile flexural walls may be found in the National Building Code....."

The chapter then proceeds to spell out the requirements for the members of ductile frames and the requirements for ductile flexural walls in addition to some general requirements.

Requirements for Ductile Frames

When classifying construction in accordance with Table II, the Canadian designer can consider a reinforced concrete frame to be a "ductile moment resisting space frame" if the flexural members, columns and beam-column connections of the frame are detailed in accordance with the requirements of Chapter 19 of CSA A23.3-1973. These requirements are essentially the same as the corresponding provisions in Appendix A of ACI Standard 318-71 [15].

Requirements for Ductile Flexural Walls

The guidelines in the Canadian concrete code for the design of "ductile flexural walls" are more comprehensive than the corresponding provisions of the ACI Code for "special shear walls". The background to the Canadian regulations is explained in the Commentary [20] of the Code and in a paper by Allen, Jaeger and Fenton [21].

High-rise structures which resist lateral loads primarily by reinforced concrete shear walls are a popular type of construction in Canada. In view of the excellent performance of well designed shear wall structures in recent earthquakes, the 1975 Canadian National Building Code permitted reinforced concrete buildings with shear walls which qualified as "ductile flexural walls" to be designed with an earthquake K factor of 1.0 (see Table II).

In attempting to present a provision which was easy to understand and apply, the 1973 CSA concrete code defined a flexural wall primarily in terms of the overall geometrical proportions of the wall. The equation defining a flexural wall was derived from the assumption that the top deflection of such a wall, when analyzed as an elastic cantilever with a horizontal force applied at the top, due to flexural deformations, should be at least ten times that due to shear deformations [20].

The inappropriate nature of a rule based on homogeneous, elastic behaviour for earthquake resistant reinforced concrete walls was pointed out in a 1975 paper by Paulay and Uzumeri [22]. In the 1977 revision [19] of the concrete code the definition of a flexural wall was changed to read "a reinforced concrete cast-in-place concrete member acting essentially as a vertical cantilever, designed and detailed so that inelastic energy dissipation takes place through flexural yielding."

The Canadian concrete code requires that the amount of vertical reinforcement concentrated near each end of ductile flexural walls be determined by calculating, (i) the area of tension steel required to resist the factored moments and axial loads given by elastic analysis, (ii) the area of tension steel required to resist the axial service load and the associated moment required to crack the wall, and (iii) the area 0.0018 $b_w d$ for grade 60 steel or 0.002 $b_w d$ for intermediate or hard grade steel and then taking the largest area required to satisfy the three requirements. If a plastic hinge is not expected to be developed in the upper half of the building, then the reinforcement concentrated at the ends of the walls may be reduced in this region to an amount given by 0.001 $b_w d$.

The vertical reinforcement near the ends of the walls must be tied in accordance with the usual requirements for columns except that in the lower half of the structure, or in regions where plastic hinges are expected to occur, the spacing of the ties must not exceed eight times the bar diameter of the vertical reinforcement.

In addition to the concentrated vertical reinforcement, the Code requires distributed horizontal steel with an area 0.0025 times the cross-sectional area of the wall and distributed vertical reinforcement with an area 0.0015 times the cross-sectional area. The maximum spacing of this distributed steel is specified as 12 in. (300 mm) in the lower half of the structure and 18 in. (450 mm) in the upper half.

In an attempt to ensure adequate shear capacity, the Code requires that each section of the wall be designed to resist a shear of:

$$V_{uc} = 1.1\ F\ V_u \qquad \text{...... (3)}$$

where V_u is the shear obtained from the analysis for the section under consideration times the appropriate load factors and F is the ratio of the calculated flexural capacity at the base of the wall to the factored moment at the base of the wall obtained from the analysis.

The area of horizontal shear reinforcement required to produce a shear strength of V_{uc} is to be determined from the usual expressions for the shear strength of beams except that in the lower half of the structure or in regions where plastic hinges are expected, the nominal shear stress taken by the concrete, v_c, is to be taken as zero.

The maximum allowable nominal shear stress on the wall resulting from the shear V_{uc} was given in the 1973 Code as $10\sqrt{f'_c}$ (in MPa this is $0.83\sqrt{f'_c}$). As there was doubt expressed, [23] and [24], about the ability of the concrete to transmit such high shear stresses across hinge regions subjected to reversed cyclic loading, the 1977 Supplement [19] to the Code reduced this value to $6\sqrt{f'_c}$ ($0.50\sqrt{f'_c}$ in MPa).

The design shear, V_{uc}, is also used in the specification of the minimum area of vertical reinforcement which must cross a construction joint. This area is given as $V_{uc}/(0.85\ f_y)$, where f_y is the specified yield strength of the steel.

Apart from the specific detailing provisions described above, the CSA Code also requires that ductile flexural walls be designed "to have adequate ductility and energy absorption capacity in accordance with generally accepted principles." The Commentary [20] on the Code suggests a simple procedure for satisfying this requirement. While pointing out that system ductility is not the same as sectional ductility and that procedures for accurately determining the required ductility of shear walls are not presently available, the Commentary goes on to suggest that as an "interim procedure" ductile flexural walls be designed so that they have a minimum sectional ductility of 3. A critical examination of this proposed simple procedure is given in the paper by Paulay and Uzumeri [22].

General Requirements

The general requirements, which may be of interest, given in Chapter 19 of the CSA Code, are listed below.

Non-continuous walls and partitions--Where a wall on a stiff partition does not continue from storey to storey, the columns supporting the wall or partition load are required to be designed for the maximum compression or tension and shear forces associated with the moment capacity of the wall or partition together with its gravity load.

Lightweight concrete--In its only reference to lightweight concrete, Chapter 19 requires that the specified concrete strength, f'_c, for lightweight concrete, must not exceed 4,000 psi (28 MPa).

Reinforcing steel--The maximum specified yield strength of reinforcement, f_y, is given as 60,000 psi (414 MPa) and it is required that the tested yield strength of the steel used not exceed the specified value by more than 18,000 psi (124 MPa).

Foundation capacity--The capacity of foundations and supports of frames and/or flexural walls is required to be sufficient to develop the total moment capacity of the frames or walls and the corresponding walls.

Moment capacity of plastic hinges--It is stated that the moment capacity of plastic hinges can be calculated by the usual provisions given earlier in the Code for determining the capacity of members subjected to flexure and axial loads.

Structural elements not part of the ductile lateral load resisting system --It is recommended that these elements be designed with sufficient strength and/or ductility so that they can accommodate a deformation three times the storey drift due to seismic forces (with load factors).

CURRENT USAGE OF THE NBC AND CSA CODES IN ERCBC

The Canadian designer, in deciding on the type of structure to employ in resisting possible seismic forces, is governed by the provisions of the National Building Code [8]. If the building is more than 3 storeys in height and is located in either seismic zone 2 or 3, the Code requires that the building have a structural system as described in Case 1, 2, 3, 4, 5 or 6 of Table II. In investigating whether a particular reinforced concrete structure satisfies the requirements of one of these six cases, the engineer is directed, by the first footnote of Table II, to the NBC Commentaries [10].

It is explained in the Commentaries that in order to be classified as Case 1, 2, 3 or 4, the reinforced concrete structure must satisfy the special detailing requirements of Chapter 19 of the CSA Code [13]. The commentary states that Case 6 applies to structures "without special provisions for ductility in the load-carrying system" and goes on to explain that "continuously reinforced concrete" as used in the definition of Case 6 "refers to reinforced concrete conforming to CSA A23.3 Chapters 1 to 18." The commentary also says "Precast concrete construction may be used in Case 6 provided the reinforcing is made continuous by means of lapped or welded splices in accordance with CSA A23.3-1973. The splices are to be encased with cast-in-place concrete."

Thus in designing say a 200 ft. (61 m) high reinforced concrete shear wall building located in zone 3, the Canadian engineer can either use a K factor of 1.0 (Case 4) and satisfy all of the requirements for a ductile flexural wall, or use a K factor of 1.3 (Case 6) and ignore the special seismic detailing rules (if the building is more than 200 ft. high and located in zone 3, the NBC requires K for Case 6 to be increased to 2.0). Similarly, if he is designing a 200 ft. high reinforced concrete frame building in zone 3, he can either satisfy all of the expensive detailing provisions of Chapter 19 and use a K factor of 0.7 (Case 1) or detail his building as he would for a non-seismic region and use a K factor of 1.3 (Case 6). An engineer interested only in producing the most economical structure will very often choose the route of higher K factors and less ductile structures.

The fact that the commentary specifically permits precast concrete structures to be classified as Case 6 has resulted in numerous pre-cast buildings being erected in high seismic risk regions in Canada.

CANADIAN RESEARCH IN ERCBC

While research relevant to the general area of earthquake resistant design is conducted in some Canadian government departments or divisions (e.g. Department of Energy, Mines and Resources and the Division of Building Research of the National Research Council) Canadian research relating specifically to the seismic performance of reinforced concrete buildings is essentially confined to the universities.

Structural engineering research in Canadian universities is primarily funded through the Office of Grants and Scholarships of the National Research Council of Canada (NRC). Table V lists Civil Engineering faculty members in Canadian universities who have recently received support from NRC for research which could be regarded as being related to the earthquake resistant design of reinforced concrete buildings. Most of the information in Table V was taken from the 1975-76 "Annual Report on Scholarships and Grants in Aid of Research" of the National Research Council [25].

The projects listed in Table V received, on average, about $12,000 per year from NRC. In comparing this level of funding with U.S. figures, it should be kept in mind that because of Canadian financing arrangements University overhead and faculty salaries can not be charged against these grants. Further numerous government scholarships are available to support superior graduate students.

Further information on the research programmes of many of the investigators listed in Table V can be found in the Proceedings of the Second Canadian Conference on Earthquake Engineering [26] and in the Proceedings of the Fourth National Meeting of the Universities Council for Earthquake Engineering Research [27].

TABLE V - CANADIAN UNIVERSITY RESEARCH RELEVANT TO ERCBC

University	Investigator	Title of Research Project
British Columbia	D.L. Anderson	Seismic analysis and design of structures
	S. Cherry	Structural response to simultaneous multi-component seismic inputs
	W.D. Finn	Soil-structure interaction
	N.D. Nathan	Behaviour of reinforced concrete frames in earthquakes
	R.A. Spencer	Behaviour of concrete structures during strong earthquakes
Calgary	W.H. Dilger	Shear strength of flat plates with special shear reinforcement
	A. Ghali	Shear strength of flat plates
Manitoba	A.M. Lansdown	A fundamental study of damping in reinforced concrete structures
Western Ontario	A.G. Davenport	The prediction of the effects of wind, earthquake, snow, etc.
	M. Novak	Vibrations of structures and foundations
McMaster	J.J. Emery	Dynamic interface behaviour of foundation-soil systems
	A.C. Heidebrecht	Earthquake engineering and dynamic response of structures
	W.K. Tso	Earthquake engineering and structural dynamics
Toronto	M.P. Collins	Structural concrete in torsion and shear
	S. Otani	Behaviour of RC columns under biaxial lateral load reversals
	J. Schwaighofer	Effects of cracks on the stiffness of shear wall buildings
	S.M. Uzumeri	Seismic resistant design of concrete structures
	G.T. Will	Analysis of reinforced concrete members subjected to load reversals
Carleton	J.L. Humar	Behaviour of framed structures subjected to seismic loading
Ottawa	N.J. Gardner	Low cycle fatigue of concrete
Concordia	C. Marsh	Earthquake forces on panelized buildings
	O.A. Pekau	Seismic behaviour of panelized buildings
École Polytech.	J. Houde	Evaluation de la résistance des structures de béton
McGill	S. Mirza	Analysis of strength and behaviour of structural concrete systems
	D. Mitchell	The design of headed stud precast connections
	B. Stafford-Smith	High-rise buildings
Sherbrooke	J.G. Beliveau	System identification in structural dynamics
Acadia	L.G. Jaeger	Dynamics of tall buildings including parametric instabilities
	A.A. Mufti	The behaviour of tall buildings under static and earthquake loads

CANADIAN RESEARCH AND DEVELOPMENT NEEDS IN ERCBC

Apart from the challenge of contributing to the advancement of the international state of the art in ERCBC, Canadian research engineers are confronted with some specific problems which arise from the present Canadian seismic design philosophy and code rules.

The Commentaries [10] on the National Building Code summarize the Canadian earthquake-resistant design philosophy in the following manner:

> "The earthquake-resistant design requirements of the National Building Code of Canada 1975 provide minimum standards which assure an acceptable level of public safety by designing to prevent major failure and loss of life. Structures designed in conformance with its provisions should be able to resist moderate earthquakes without significant damage, and resist major earthquakes without collapse. For the purpose of this section, collapse is defined as that state which exists when exit of occupants from the building has become impossible because of failure of the primary structure."

Before this design philosophy, which is generally accepted, can be used to derive specific design criteria, answers are needed to the following general questions:

A) For the various regions of Canada what are the ground motion characteristics of "moderate" and "major" earthquakes? Do they correspond to 50 year, 100 year, 200 year or 400 year return period earthquakes?

B) What is the desired performance of buildings when they are subjected to these various earthquakes?

C) What are the specific structural design criteria needed to ensure that structures achieve the desired level of performance?

A more detailed listing of some of the Canadian problems, and hence research needs, which result from trying to answer these three general questions in the context of ERCBC is presented below.

1. Use of 100 Year Peak Ground Acceleration

The work of Milne and Davenport [5] and the ongoing work by the Department of Energy, Mines and Resources (EMR) of the Government of Canada [1][16] attempts to quantify for the use of engineers the characteristics of various return period earthquakes. Based on this work, tables of peak horizontal ground acceleration at various localities for different probabilities of annual exceedance are given in the Commentaries [10] to the Code. Further, it is stated that other probabilities of exceedance for any site in Canada can be obtained from EMR.

Table VI, which lists peak ground accelerations with 0.01 and 0.005 probability of annual exceedance for some Canadian and U.S. localities, may assist the participants of the workshop in calibrating Canadian practice. The 0.01 and 0.005 values are also called the "100-year" (A_{100}) and "200-year" (A_{200}) acceleration levels.

TABLE VI - PEAK GROUND ACCELERATIONS [7] [10]

Locality	A_{100} (% gravity)	A_{200} (% gravity)
La Malbaie	49.5	114.0
Quebec City	7.1	12.4
Montreal	3.6	5.6
Ottawa	4.8	7.9
Toronto	2.7	4.5
Vancouver	8.0	16.9
Victoria	11.1	23.4
San Francisco	20.5	50.0
Los Angeles	16.0	34.0
San Diego	11.0	20.5

The Code [8] requirement that the A_{100} value be used as the assigned horizontal design ground acceleration in the dynamic analysis approach would seem to imply that the 100 year earthquake is the "major earthquake" which the structure must resist without collapse. When it is recognized that a building with a 50 year life has about a 40% chance of experiencing ground accelerations greater than A_{100} and about a 20% chance of experiencing ground accelerations more than twice as great as A_{100}, then serious doubts are raised about the choice of A_{100} as a design basis.

Part of the rationale for the choice of the 100 year earthquake was the apparently comparable effects of the "100 year" acceleration level and the "30 year" wind speed employed elsewhere in the Code [6]. Figure 5, which is reproduced from a paper by Rainer [27], illustrates some of the difficulties involved in comparing wind loads and seismic loads. The magnitude of the "30 year" wind load appears to be a large fraction of the maximum possible wind load. That is, if we visualize extrapolating the plot to say 400 years, the wind load would only exceed the design value by about 60%. The 100 year seismic load, on the other hand, is only a small fraction of maximum possible seismic load. Extrapolating the plot to 400 years would seem to indicate a seismic load more than 300% in excess of the design load.

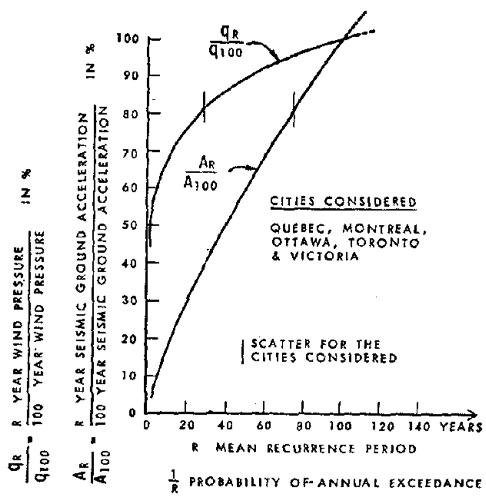

FIGURE 5 : COMPARISON FOR STATISTICAL PROPERTIES OF WIND AND EARTHQUAKE LOADING [27] [6]

It would seem that more work is required before an acceptable definition of what constitutes a "major earthquake" can be obtained.

2. Structures with Low Ductility in High Seismic Risk Zones

The fact that the Canadian Code [8] permits concrete structures with only "nominal ductility" to be constructed in high seismic risk zones raises a number of questions that need further study.

- Does increasing K from 1.0 to 1.3 provide shear wall structures having no special detailing provisions with the same ability to "survive" earthquakes as that of the ductile flexural wall structures?
- Does a reinforced concrete frame which has no special joint reinforcement and which only has nominal ties in the columns have a reasonable chance of "surviving" a major earthquake if it has been designed to be strong enough to resist a lateral force corresponding to a K value of 1.3?
- Will precast concrete structures or flat plate structures designed to resist a lateral force corresponding to a K value of 1.3 "survive" a major earthquake?

It would seem to the writers of this report that because of the difficulty in accurately predicting the actual magnitude of possible earthquakes in zone 3 concrete structures with only "nominal ductility" should not be constructed in such zones.

3. Protection of Post-Disaster Service Buildings

As stated in the Commentaries [10] to the Code, "some structures are designed for essential public services and it is imperative that these structures be operative after an earthquake." In an attempt to achieve this objective, the Code requires that the design earthquake force for such structures be increased by a factor of 1.3.

At first sight increasing the design earthquake forces by 1.3 would seem to be comparable to the specified practice for wind which is to use a 100 year wind for buildings required for post-disaster services rather than the usual 30 year design wind. As can be seen from Figure 5, the 100 year wind force is about 1.3 times the 30 year wind force. For wind loading, increasing the design force by a factor of 1.3 reduces the chance of an essential building experiencing a greater than designed for load to about 30% of that of a normal building. For seismic loads, on the other hand, increasing the design force by a factor of 1.3 only reduces the chance of overload for essential structures to about 75% of the chance for normal buildings.

Increasing the design force by 30% will presumably decrease the ductility demand by about the same amount. For a reinforced concrete frame building the remaining ductility demand will still result in very substantial inelastic deformations during a major earthquake. Will such deformations render the structure, or the essential equipment housed therein, inoperative? In order to protect essential buildings will it be necessary to establish specific acceleration, displacement and velocity limitations for such structures?

4. Comparison of Static and Dynamic Design Procedures

The fact that the static and dynamic procedures of the Code [8] do not give the "similar results" for "regular buildings" predicted by the Commentaries [10] has caused considerable concern to a number of designers.

For structures with long periods the dynamic procedure predicts much smaller base shears than the static procedure while for short period structures the reverse is true (see Fig. 4). Though the forthcoming 1977 NBC revision will remove some of the possible effects of this inconsistency (it will require that the base shear from the dynamic analysis not be taken as less than 90% of the base shear from the static analysis) much more work is needed to develop rational procedures which will not display such inconsistencies.

As it has been agreed that the dynamic base shears for long period structures are too low, does it follow that the static base shears for short period structures are too low? In other words, are short period structures designed by the static approach more susceptible to earthquake damage than long period structures designed by the static approach?

5. Period Estimation and Design Seismic Force

The seismic design force used in the static procedure of the Canadian Code [8] is assumed to be a function of the fundamental period of the structure, T. Empirical expressions are given to evaluate the period but it is stated that "where technical data proves otherwise" the designer may use other values for the period. The Commentaries [10] make clear that what is required is to "determine the period T for a structure by more refined methods of calculation."

A dynamic analysis of a skeletal structure usually results in estimating a longer period for the structure than that given by the empirical formulae. The longer period would mean that the structure could be designed for a smaller seismic force (a 20% reduction would be typical). Is such a reduction of seismic design forces justifiable?

Should the base shear coefficients be related to the initial stiffness (including the effects of non-structural elements), the uncracked stiffness of the skeletal structure, or to the effective stiffness of the yielding structure?

6. Required Shear Capacity of Ductile Flexural Walls

The current Canadian concrete code [13] requires a ductile flexural wall to be designed to resist a shear of 1.1F times greater than the shear obtained from analysis. The term F is a scaling factor which increases the predicted shears and moments such that the moment at the base of the wall equals the calculated flexural capacity of the wall. As the actual flexural capacity of the wall might be considerably greater than the calculated value (strain-hardening of the steel and probable material overstrengths are not considered in the suggested calculation procedures) and as the actual ratio of shear to moment that occurs in the wall might be much higher than the predicted ratio (Bertero et al [28] have demonstrated that the actual V/M ratio can be more than 60% greater than the code predicted ratio) it seems certain that a factor larger than 1.1 should be used to determine the required shear capacity. Work is needed to determine an appropriate expression for the needed shear capacity.

7. Use of Lightweight Concrete in Ductile Structures

By specifying a maximum compressive strength for lightweight concrete Chapter 19 of the Canadian concrete code [13] permits the use of lightweight concrete in the construction of ductile space frames and ductile flexural walls. It seems unlikely that the only precaution needed when using lightweight concrete is to keep f_c' below 4000 psi (28 MPa). Work is needed to understand the behaviour of lightweight aggregate concrete members subjected to load reversals and to investigate what modifications need to be made to the detailing rules for ductile space frames and ductile flexural walls to ensure that these rules work equally well for lightweight concrete.

8. Yield Strength of Reinforcing Steel

The Canadian concrete code [13] requires that the yield strength of steel used in structures designed according to Chapter 19 should not exceed the specified strength by more than 18 ksi (124 MPa). Evidently the only grades of Canadian steel that have any chance of being reasonably close to the specified strengths are Grade 40 and Grade 60 (Grade 50 seems to be a "catch all" steel with great variations in its mechanical properties) and even for these grades the steel producers claim that the 18 ksi limit cannot be met.

Figure 6(a), which is reproduced from a private communication by J.G. MacGregor [29], shows the yield strength distribution obtained from 249 tests of Canadian and U.S. Grade 40 bars, #3 to #11 in size. The mean yield strength is 48.8 ksi and the standard deviation is 5.2 ksi. Figure 6(b) (also from MacGregor) shows a similar distribution from 273 mill tests of Grade 60 bars. The mean yield strength from this distribution is 71 ksi and the standard deviation is 6.6 ksi.

It can be seen from Figure 6 that there are difficulties in meeting the 18 ksi limit and that there would be even more difficulties in meeting a proposed upper limit of 1.25 f_y.

In this regard, it is of interest to note that a new Canadian grade of weldable bar (G30-16M) is now proposed. This grade will have a specified yield of 400 MPa (58 ksi) and a specified maximum yield strength of 540 MPa (78 ksi). That is the maximum yield strength will be 1.35 f_y or 140 MPa (20 ksi) above the specified yield.

Further work is needed to produce workable specifications for the allowable range of yield strengths to be used in ERCBC and to study the effects of changing these specifications on the detailing requirements for ductile structures.

9. Limit States Design and ERCBC

Work will be needed to study the implications for the earthquake resistant design of reinforced concrete buildings of going to the limit state design philosophy of the Canadian Code [8].

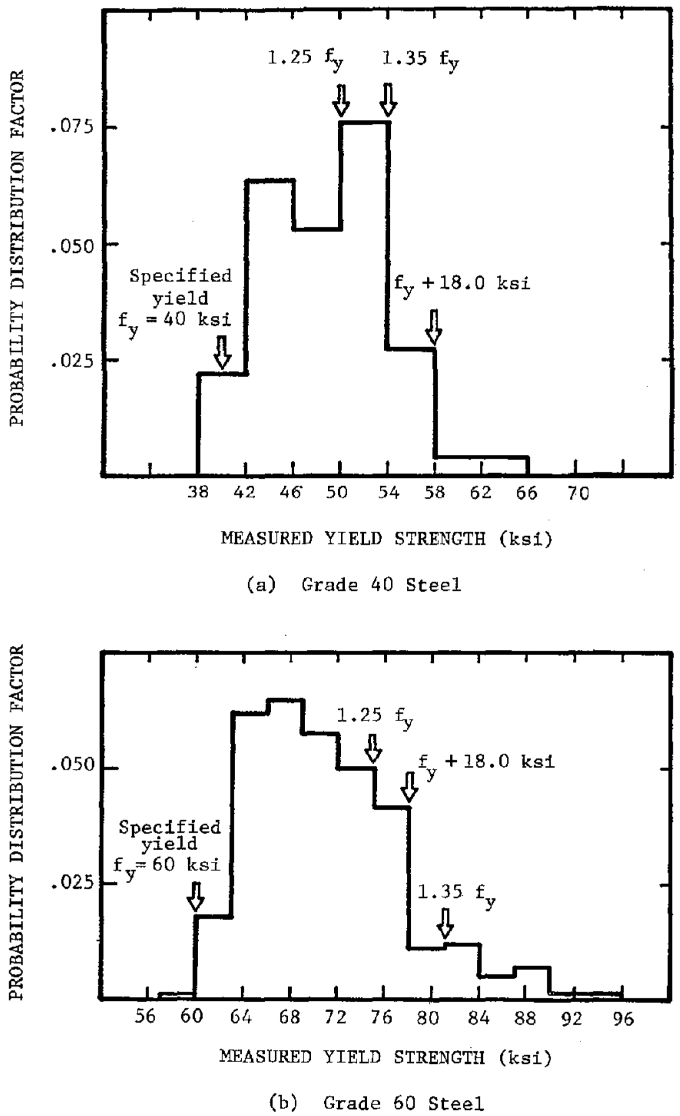

FIGURE 6: DISTRIBUTION OF MEASURED YIELD STRENGTH OF REINFORCING STEEL
(After MacGregor [29])

For example, if the role of the load factor is to account for the possibility that the actual load may exceed the design load, then is it reasonable to use the same factor, 1.5, for wind loads and for earthquake loads? For wind loads (see Fig. 5) it may be very unlikely that the actual load will be greater than 1.5 times the "30 year" design load but for earthquakes the chances are much higher that the actual "load" will exceed 1.5 times the "100 year" design "load". How will the concept of ductility be fitted in to the limit state design?

10. Compatibility of Canadian and U.S. Practice in ERCBC

When the Canadian and U.S. seismic zoning maps for border regions are compared, some marked differences are evident. While it would be interesting to hear seismologists explain the rationale behind these differences, as structural engineers what is of more concern to us is that similar adjacent buildings located on either side of the border should have probabilities of failure consistent with the design philosophy of each country. It is conceivable that Canada may accept a different probability of failure for a building in Niagara Falls, Ontario, than for its identical twin in Niagara Falls, New York. This may be due to different social priorities or risk taking philosophies. However, what is desirable is that this difference is the result of conscious decisions.

The development of some bench mark buildings to calibrate Canadian and U.S. design provisions would be very useful. Some preliminary discussions on this needed work has already taken place between CANCEE and the Applied Technology Council. Furthermore, CANCEE is proceeding to establish a number of Canadian bench mark structures which can be used to test various code change proposals and to calibrate existing code procedures [30].

CONCLUSIONS

While the art of designing and constructing earthquake-resistant reinforced concrete buildings was greatly improved in Canada over the last 20 years there are still a number of areas in which concern can be expressed (e.g. non-ductile structures in zone 3, survival of essential structures, choice of the 100 year earthquake). The question of whether these possible deficiencies in design practice result in reinforced concrete structures possessing inadequate seismic resistance can be answered in one of two ways. We can either wait for a major earthquake, or try to answer the question by means of co-ordinated analytical and experimental research.

Further research will hopefully advance the state-of-the-art to the point where code committees will be able to write logical, comprehensive and simple rules for the design of ERCBC. However, on the way to this stage code writing authorities must resist the temptation to rationalize separately the various component parts of the seismic design procedure. What must always be kept in mind is the effect of a particular change on the final completed structure.

"Correcting", or making "more precise", a few of the parts of the traditional procedure for seismic design without evaluating the effects of the changes on the total design may be counter productive.

ACKNOWLEDGEMENTS

We gratefully acknowledge the help of our Canadian colleagues who contributed to our appreciation of these complex issues by their publications and by spending many hours of their time in discussion of these issues with us. If in any areas we misunderstood them, we apologize. We also gratefully acknowledge the invaluable assistance received from Mrs. Agnes Kaneko in preparing this report.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

A EUROPEAN VIEW ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION

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1 - INTRODUCTION

Reinforced concrete building construction should be viewed as a whole. Earthquake-resistant design should be one part of the whole compatible with the other parts. The fundamental concepts of reinforced concrete building construction should be the same as other types of construction.

In Europe,due to the joint effort of several international associations, a system of unified standard codes of practice for structures is at an advanced stage of preparation. This system consists of a set of volumes.

Volume I [1] , prepared by the Joint-Committee on Structural Safety, presents common unified rules for different types of construction and material. It is the aim of this volume to formulate and to implement modern concepts of structural safety. It represents a stage of progress which gives a firm support to specific design rules within the framework of the unified rules. Its adoption shall create the desirable unity of basic concepts to be followed in the different codes.

Volume II [2] , prepared by the Euro-International Committee for Concrete, CEB, presents a code of practice for the design and execution of concrete structures. It consists of a set of rules summarizing present knowledge on structural behaviour of reinforced and prestressed concrete elements and structures. It extensively covers behaviour under monotonic loading, makes some reference to fatigue problems but ommits repeated loading of the type related to earthquake-resistant design.

Volume III [3] , prepared by the European Convention for Constructional Steelwork, covers the design and execution of steel structures.

Volume IV deals with mixed reinforced-steel structures and is in preparation by a Joint-Committee appointed for this purpose.

Volume V, on Masonry Structures, and Volume VI on Timber

Structures are being drafted by CIB working commissions.

In all these documents the problem of earthquake-resistant design is almost neglected. Appendix III of Volume I which presents methodologies for the definition of variable actions dedicates some very brief comments to seismic actions.

In Volume II leave is given to consider seismic actions as a variable loading, to be combined in the usual way with other types of loadings, or as an accidental loading. In both cases no detailed design rules are given. However the CEB has for several occasions expressed the intention to deal with problems of earthquake-resistant design in the near future.

A different approach to these problems is followed by the European Committee for Earthquake Engineering. It created a Working Group on Unification of European Countries' Codes which drafted a Unified European Code for seismic design in seismic regions [4] . This draft is of the traditional type. As usual in earthquake resistant regulations it adopts the concept of seismic coefficient, and by splitting it into several parameters takes into account different situations and influences. There is no specific part dealing with reinforced concrete design. The International Federation for Prestressing, FIP, also has been concerned with seismic regulations and published the draft of a general code on the subject [5] .

The present situation concerning earthquake resistant regulations in European countries is analogous to the situation in countries of other regions [6]and[7] . European countries try to improve their codes on a national basis; there is not much international cooperation. There is little coordination between earthquake-resistant regulations and design codes that do not deal with seismic actions. The definition of seismicity and of seismic actions does not benefit from recent advances in these fields. The problems of combination of actions, definition of limit--states, behaviour and repeated loading are not treated in a rational ways. Design and execution specifications do not make full use of existing knowledge. There is much to be done which by no means is peculiar to Europe. International cooperation on a world-wide basis is a need no longer hindered by distance.

Although there is much to be done, it should be recognized that earthquake-resistant regulations as they exist have played and are playing a very important rôle in the saving of lives and in the reducing of damage all over the world.

If can be argued what is more important: to implement and to enforce existing regulations or to improve them. However these two problems have little in common. The best solution is to motivate the bodies responsible for these two types of actions to operate in the most efficient way in both fields.

Points of view are expressed concerning the improvements needed in the definition of basic concepts and in the design of reinforced concrete buildings.

2 - IDEALIZATION OF SEISMIC ACTIONS

2.1 - General — Nowadays it is unnecessary to justify the use of probabilistic formulations in structural codes. Basic variables representing actions, dimensions and mechanical properties should be considered as random variables.

One of the aims of the Joint-Committee on Structural Safety consists in improving the definition of actions. For this purpose it has published a set of basic notes on actions [8] which include not only a basic methodology but also specify idealizations of the most important types of loadings: dead load of concrete structures; superimposed loadings in dwellings, office buildings, retail premises and car parkings; snow loads on roofs; wind velocities and seismic vibrations.

Idealizations of seismic actions used at present in most codes deserve much criticism. Even the most advanced ones do not have a sound probabilistic basis. This criticism does not mean that codes should be more involved than they are. It is accepted that in usual situations seismic actions may by represented by seismic factors defined in a global way. What is imperative it that the seismic factors be a convenient representation of real seismic actions, it being stated under which assumptions they are derived and within what limits they should be applied.

Elastic response spectra are a powerfull concept directly related to the seismic vibrations. However, its field of application is also limited. The present tendency to generalize them by defining inelastic response spectra [9] is not the only way to deal with the problem. Inelastic response spectra give a remote idealization of soil vibrations.

In the basic note on seismic vibrations [8] the basic variables which represent earthquake actions are obtained by combining two idealizations: one dealing with the occurrence of earthquakes, the other concerning the description of seismic vibrations.

The convenience of splitting the idealization of the occurrence of earthquakes and the idealization of their description imposes a link between the two idealizations. This link may be expressed by one or more measures of intensity. It is not imperative to fix a single measure. The most important thing is to be able to relate the different measures, to understand their meanings, and to judge the convenience of their use.

2.2 - Earthquake intensity — In modern seismology the wave radiating from the source of an earthquake is described by parameters such as moment, length and stress drop [10] . However, the different types of magnitude are still the most usual measure of the energy content of earthquakes.

At a given location distinction should be made between bedrock seismic vibration and vibrations at the surface. In this way it becomes easier to relate the parameters which describe the earthquake at the source and the earthquake at a given location.

All these concepts were dealt with by more than one hundred papers presented in topic 2 (Ground motion, seismicity, seismic risk and zoning) at the Sixth World Conference on Earthquake Engineering. Cornell in his summary of the papers [11] emphasizes the need to have a single scalar parameter to define the strength of a motion. It is often indicated by some authors that this parameter should measure the damage done to structures by earthquakes.

This aim is not fundamental and it cannot be satisfactorily reached due to the diversity of structures. In the present context the need to define earthquake intensity derives from the need to link occurrence and descriptive models.

For the said purpose any parameter measuring the power content of the most intense part of the accelerogram would be satisfactory. Under simple assumptions this power content can be related to the peak acceleration. Consequently, it is suggested that earthquake intensity be expressed by a nominal value of the peak acceleration. The determination of this nominal value should be discussed in detail to take into account: instrumental corrections; three dimensional aspects of the vibration; correlations with other parameters, such as, Housner or Arya's intensities, ordinates of Fourier spectra and the integral of the power spectral density of acceleration. This last combination is particularly important and simple.

Assuming the seismic vibration to be a sample of a stationary Gaussian process of power spectrum S (f) , the mean of the peak values of acceleration (for different samples) is given by

$$a = \mu \sqrt{\int_0^\infty S(f)\, df} \quad \ldots\ldots\ldots\ldots \quad 1)$$

where μ is a parameter which depends relatively little on the duration. For a duration of 30 sec $\mu \approx \sqrt{10}$ [8] . For other durations the values of μ can be easily deduced [12] .

Studies leading to a standardization of the instrumental definition of intensity are strongly recommended. These studies should be complemented by the definition of correlations to other instrumental and subjective scales used at present and in the past.

2.3 - Occurrence of earthquakes – Seismic recording at a site allows to determine the maximum peak accelerations which occur each year. This information can be complemented by deriving peak accelerations from observed magnitudes and location of sources of past earthquakes, magnitudes being transformed into peak accelerations by means of attenuation formulae. Intensity of earthquakes measured by subjective scales can further be introduced and combined with geological and geotechnical data. Finally generation models are also useful in deriving the probability distribution of maximum annual values of peak accelerations.

The annual extremes of magnitudes in a region being represented by an extreme probability distribution of Type I leads to probability distributions of maximum annual values of peak accelerations of Type II [13] . Fig. 1 shows the Type II distribution

$$F_{II}(a) = \exp\left(-(k\,a)^{-\beta}\right) \quad \ldots\ldots\ldots\ldots \quad 2)$$

with $\beta = 2$. The exponent $\beta = 2$ applies in several regions of the world.

Experimental data and physical considerations justify a tendency towards truncature of the probability distribution of annual extremes in its upper tail. This upper tail is particularly important for design purposes if a reference period of the order of 50 years is adopted.

To estimate the peak acceleration value which corresponds to a return period of 50 years, it is acceptable to use the fitting for a Type II distribution. This is no longer the case when estimating maxima in 50 years.

The solution which is suggested, Fig. 1, consists of substituting above the 0.98 fractile the Type II distribution of annual maxima by a Type I distribution:

$$F_{I}(a) = \exp\left(-\exp\left(-\alpha\,(a-u)\right)\right) \quad \ldots\ldots\ldots\ldots \quad 3)$$

The condition of both distributions Type I and Type II being tangent at the 0.98 fractile is imposed.

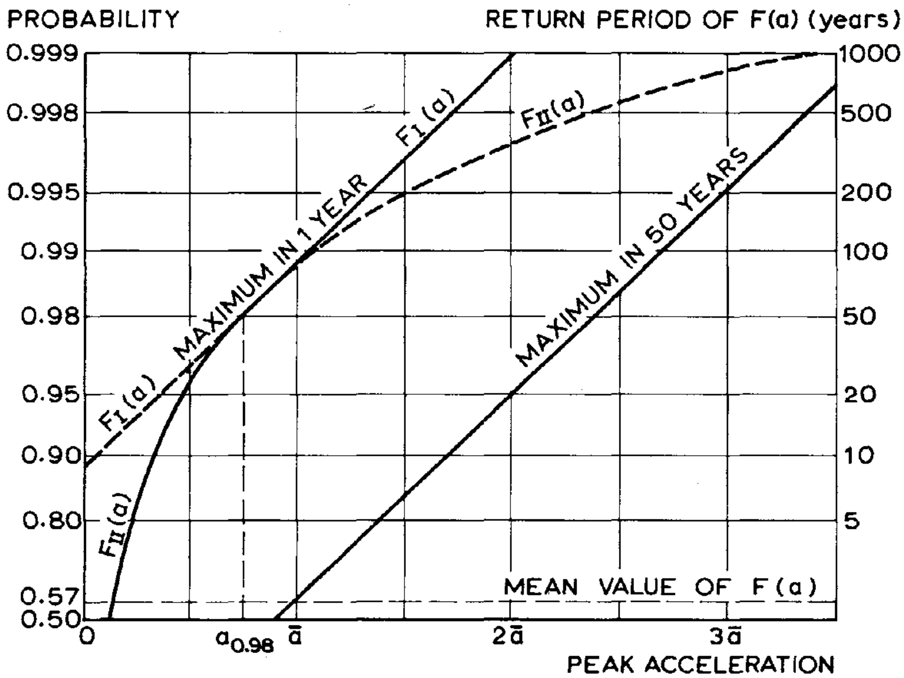

Fig. 1 - Probability distributions of maximum peak accelerations in 1 and 50 years.

Under these assumptions the distribution of maximum accelerations in 50 years can be easily obtained. Near and above the mean value this distribution is well represented by a Type I extreme distribution with the mean value $\bar{a} = 1.3\ a_{0.98}$ and the coefficient of variation 0.5.

The solution adopted is controversial. There is very litle information allowing to estimate the type of distribution of extremes in 50 years and its coefficient of variation. However the consequences on design which derive from these assumptions are important.

Studies leading to improved definition of the probability distributions of extreme values of peak accelerations in periods of 50 years are strongly recommended. Further studies on the idealization of the occurrence of earthquakes may also be useful particularly if they are carried out bearing in mind the needs of the probabilistic methods of structural design.

2.4 - Idealization of seismic vibrations – To idealize seismic vibrations it seems necessary to distinguish between descriptive and design models. The aims to be attained by these two types of models are different. Descriptive models should idealize the vibrations in order that the fundamental features of one occurrence are kept. On the contrary design models shall be chosen in a pragmatic way. Structures designed on basis of design models will behave on an optimal way under a set of possible circumstances.

For an earthquake defined by its source parameters it is not yet possible to obtain univoque descriptive models of the vibration in the near and far fields.

An overall appreciation of existing records in epicentral regions [14] shows two main types of shocks: i) shocks which include a dominant cycle, ii) shocks which do not include a dominant cycle.

A large percentage of far field accelerograms and many near field ones do not include a dominant cycle. In this case the convenient descriptive model should be based on random vibrations. Most refined models consider the random vibration as non-stationary, both in intensity [15] and in frequency content.

The far field accelerograms of the Romanian earthquake of March 1977 shows in one of its components a single cycle whose amplitude is about three times the amplitude of the most intense part of the vibration. Accelerograms of similar type have been recorded in several other circumstances.

For simplifying the design process it would be convenient to adopt a single type of design model. It is not yet clear if this model should be of the single cycle type or random vibrations of different durations.

The basic note on actions dealing with seismic vibrations [8] represents bedrock and surface vibrations by a stationary Gaussian process of 30 second duration and zero mean value of acceleration. According to the types of soil, the power spectral densities of acceleration S (f) take the shapes indicated in Fig. 2.

The shapes of these power spectra were selected taking into account that they should be used as design spectra and not as descriptive models. Descriptive models should have had a shape closer to that of the transfer function of a one degree-of-freedom oscillator.

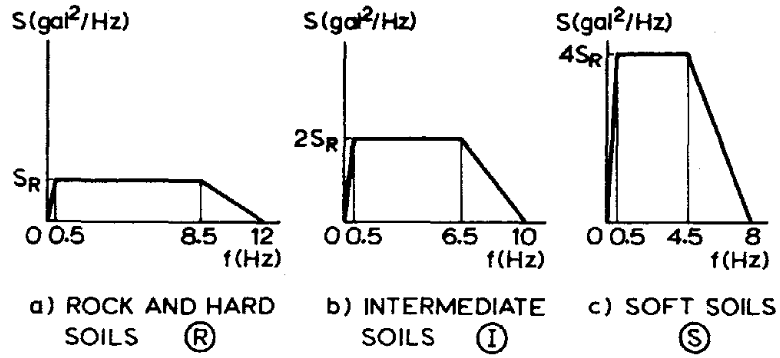

Fig. 2 - Shapes of power spectral densities of acceleration.

As indicated in [16] it is possible to compute the response spectra which corresponds to the power spectra represented in Fig. 2. Fig. 3 shows these response spectra when a bedrock maximum acceleration a = 100 gal and a fraction of critical damping μ = 0.05 are adopted.

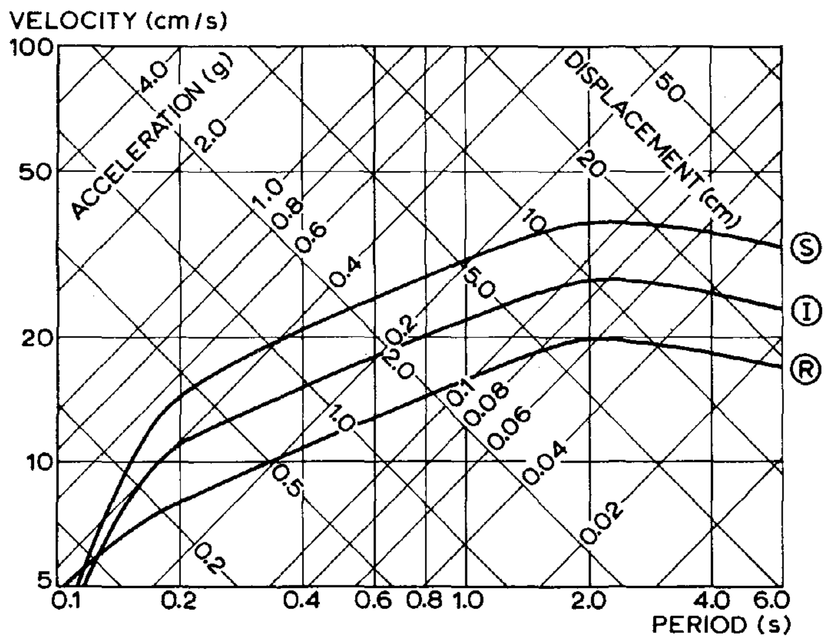

Fig. 3 - Response spectra for a = 100 gal and μ = 0.05 which correspond to the power spectra indicated in Fig. 2.

The programs available at LNEC also make it possible to transform response spectra into power spectra.

The above considerations refer to horizontal vibrations in one direction. Idealization of vertical and rotational components and correlations in time and space should have been studied.

The most convenient methodology to idealize seismic vibrations is an involved problem which should be solved by a cooperative work of those directly interested in the problem. This task should be fulfilled having in mind the fundamental concepts of probabilistic design of structures.

3 - FORMULATION OF STRUCTURAL SAFETY

3.1 - Definition of limit states – The Common Unified Rules for Different Types of Construction and Material [1] , classify limit states in two categories:

a) the ultimate limit states;
b) the serviceability limit states.

It is mentioned that ultimate limit states may be reached due to: loss of equilibrium; rupture of critical sections of the structure or excessive deformation; transformation of the structure into a mechanism; buckling; fatigue. The pertinence of this classification is questionable. However, it emphasizes different aspects of failure.

To obtain a rational design it is particularly important to discuss how ultimate limit states are reached under seismic actions.

Consider the simplest case of a reinforced concrete column, Fig. 4. Under alternative cycles of horizontal displacements applied at the top,

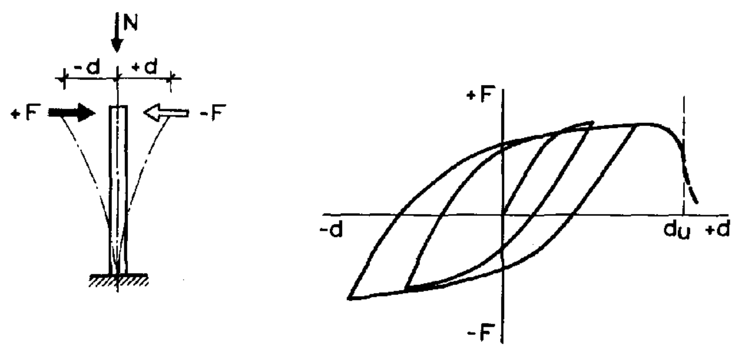

Fig. 4 - Reinforced concrete column under alternative repeated cycles of imposed displacement.

the column behaves as indicated in the figure. The ultimate limit state is attained when the rotation capacity of the column is reached, this corresponds to an ultimate displacement, d_u .

In the above case to define the ultimate limit state it is necessary to define ultimate displacements and not ultimate forces. The values of F are only auxilliary and could be omitted. In this simple case the checking of safety for seismic actions should be carried out by comparing ultimate displacements and the maximal displacements produced by the seismic vibrations. It is meaningless to speak about resisting and acting forces.

On the contrary, a bridge beam fails when the load of the truck which crosses it exceeds the ultimate force that the beam can resist. In this case the checking of safety should be carried out comparing the ultimate forces and the maximal forces applied by the truck.

Usual situations are much more complex than the two simple cases indicated above. Permanent loads and variable actions of different nature combine and act simultaneously. Limit states are not reached under a monotonic variation of the actions but under repeated variable cycles. Damage (or utility) varies continuously as a function of the intensity of the actions and can only be expressed approximately by discrete limit states.

Failure criteria in Earthquake Engineering are discussed by Bertero and Bressler [17] in a contribution to a panel discussion at the VI World Conference in Earthquake Engineering. In this paper the concept damage ability limit state is introduced. A classification is presented of the principal ways along which ultimate limit states are reached.

The phenomena associated with variable repeated excitations are classified as: long-endurance fatigue; low-cycle fatigue and incremental collapse, the two last ones being of particular interest for seismic actions. It is indicated that the real danger in low-cycle fatigue is not fracture of the structural material but deterioration of the stiffness.

The point of view seems to oversimplify the problem. Under variable repeated actions ultimate limit states may be reached under as large variety of circumstances: limit deformability of steel in tension, crushing of concrete, buckling of longitudinal bars, failure of transverse bars, deterioration of bond, change of geometry of the structure, local and overall buckling, etc.

These different phenomena should be analysed and understood. The information gathered by testing complex structures, elements and connections, although valuable, will never be sufficient to understand the seismic behaviour of concrete structures.

3.2 - Theorization of structural concrete — A satisfactory understanding of the behaviour of structural concrete will only be obtained by a theoretical support which allows to forecast the main features of this behaviour. The basic information for this theory shall derive from the idealization of the mechanical properties of concrete and steel under variable repeated loading [18] . Furthermore, bond between steel and concrete plays an important rôle in the overall behaviour of reinforced concrete members. A large variety of phenomena, such as adherence, friction, identation and pulling out is included under the general terms of bond. The basic aspects of all the phenomena should be investigated.

The behaviour of steel bars is influenced by the surrounding concrete both in tension and in compression. An understanding of this behaviour is necessary for deriving the behaviour of more complex elements.

A simple bar surrounded by concrete presents in tension a complex behaviour which is idealized in Fig. 5. For monotonic loading the stages: non-cracked, non-stabilized cracking, stabilized cracking and yielding are identified. Under variable repeated loading the paths indicated by arrows are followed.

An analogous situation occurs in elements under compression formed by: concrete core, concrete cover, longitudinal and transverse bars, Fig. 6. In this case the overall behaviour can be obtained by associating in parallel the behaviours of the core, of the cover and of the longitudinal bars.

Other models have been derived to explain the behaviour in shear, torsion, bond and combined load-effects. These theoretical models should describe the relationships between load-effects and generalized displacements and allow to obtain the ultimate deformability of elements and structures, both in monotonic loading and in alternative repeated cycles.

A further difficulty derives from the fact that the ultimate resistance to a component (e.g. axial force) is often reduced by large alternative repeated cycles of other component (e.g. bending). Consequently it is necessary not only to check the ultimate deformability of the structure but also its carrying capacity at the deformed state. This influence may be amplified by the P-Δ effect.

3.3 - Seismic response — A very large percentage of earthquake engineering studies deals with the response of structures acted on by seismic vibrations. The vibrations being idealized by stochastic processes, the quantities which describe the response are random variables. The

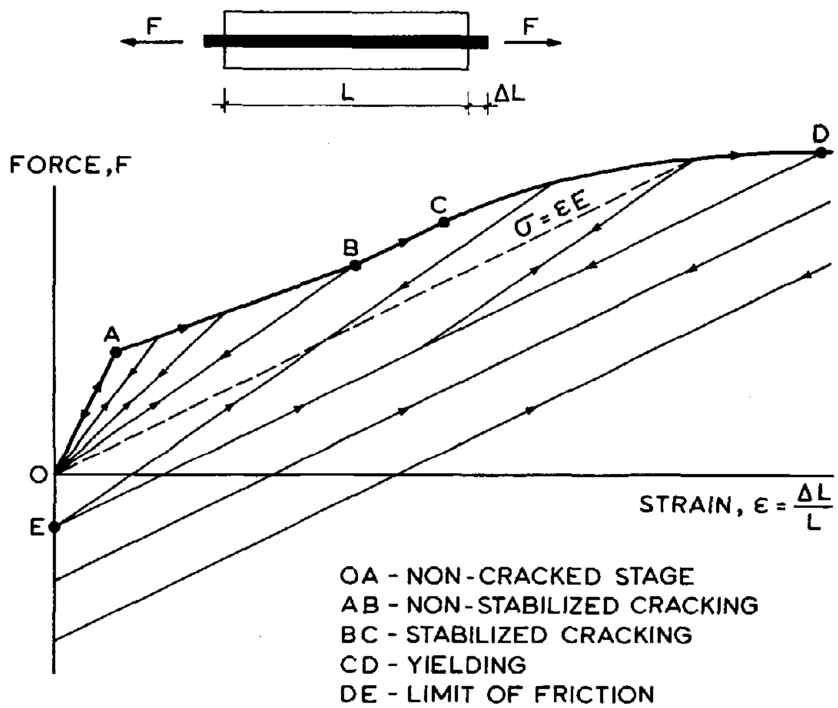

Fig. 5 - Force-strain relationship for steel bar embedded in concrete under repeated tension.

extreme values of the response are well idealized by extreme distributions. Probabilistic studies on structural response should conclude by presenting the type and the parameters of the extreme distributions of the response.

However, this is not enough for a rational probabilistic design. Usual design rules consist in limiting the probability of attaining ultimate limit states during a reference interval of time. To be able to compute this probability it is necessary to compute first the convolution of the distribution of occurrence of earthquakes and of the distribution of extreme response [19] . In this way the probability of attaining a given response in a reference interval of time is obtained.

As indicated in 2.3) the coefficient of variation of the extreme peak acceleration in 50 years is about 0.5. The coefficient of variation of structural response very much depends on the duration of the earthquake,

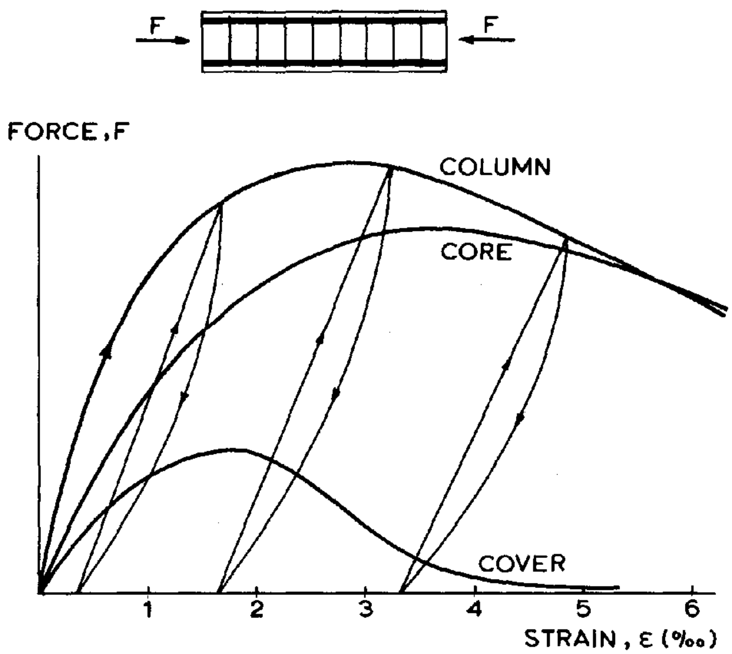

Fig. 6 - Force-strain relationship for reinforced concrete column under repeated compression.

the amount of non-linear behaviour and on the natural frequency of the structure. As shown by Murakami and Penzien [9] the coefficient of variation of linear response increases from 0.1 to 0.5 as the duration of the earthquake decreases from 30 to 0.5 second. In non-linear behaviour, coefficients of variation even higher than 1.0 may be reached in particular for structures of high natural frequency (in the range of 5 to 10 Hz).

Consequently, the distributions of the extreme values of the response during 50 years shall always present very high coefficients of variation, which are considerably higher than for any other usual type of loading. This fact has important effects in practical design.

3.4 - Safety checking – The general rule for safety checking consists in verifying if the probability of surpassing a given limit state during a reference interval of time is sufficiently small. This checking may be performed in different spaces of variables: the space of basic variables (input-space), the space used to define the limit state (output-space) or any other space of variables obtained from a transformation of the two (state-space) [20] .

The space where it is most convenient to operate depends on the type of problem to be solved. This question deserves particular attention in earthquake engineering due to the fact that direct loads and imposed variable deformations act simultaneously.

Imposed deformations being paramount, the limit states should be expressed by ultimate displacements. Consequently, it would be advisable to carry out the safety checking in a space which would include generalized displacements as variables. The safety checking would consist in comparing extreme displacements due to earthquakes to ultimate displacements (limit states), both defined in probabilistic terms.

The main difficulty to proceed along these lines derives from the scarce information on limit-state displacements, reached both in monotonic and repeated loading. This is the problem about which existing information is more unsatisfactory and one of those on which research should concentrate. Without this information a full rational probabilistic design cannot be established. The introduction of the concept of ductility factor is a deviation with many drawbacks.

It is to be expected that the distributions of limit-state displacements present coefficients of variation much higher than those of limit--state load effects. Consequently, both the coefficients of variation of the response and of the limit-states shall be very high as compared to the usual ones.

In terms of displacements, the probability of reaching a limit-state is obtained by computing the convolution integral of the distributions of response and limit-state displacements. In practice this may be simplified in Level 2 and Level 3 methods by checking a reliability index or imposing partial factors of safety, respectively [1] .

Typical results of this basic problem [19] show that when combining extreme distributions of high coefficient of variation it is uneconomical to obtain probabilities of failure smaller than about 10^{-3}. Therefore, in seismic zones usual values of the probability of failure of the order of magnitude of 10^{-5} to 10^{-6} are difficult to implement.

Studies to quantify the seismic risk which corresponds to present building techniques and to indicate which changes should be introduced in order to get satisfactory protection against earthquakes are highly recommended.

4 - CONCLUSIONS

Structural design should take full advantage of a perfect link of basic concepts and information in all their pertinent aspects. This aim may only be reached by a world-wide cooperative work in the different fields.

Much progress has been achieved recently. However, practice has benefitted little from all this progress.

Points of view on the directions along which research should proceed were indicated. It is recognized that several important subjects were omitted, particularly those dealing with execution and control.

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A REVIEW OF RECENT RESEARCH IN JAPAN AS RELATED TO THE EARTHQUAKE RESISTANT DESIGN OF REINFORCED CONCRETE BUILDING STRUCTURES

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INTRODUCTION

In the long history of earthquake resistant design of reinforced concrete building structures in Japan, the most remarkable developments have been made in the last decade, stimulated by the experience of structural damages caused by the Tokachioki earthquake of 1968. The Architectural Institute of Japan published a book entitled "Earthquake Load and Earthquake Resistance of Building Structures" [1] in Jan. 1977, compiling the results of most recent coordinated activities of many structural committees of the Institute. In this publication a new concept of the "Earthquake Load" was proposed, and it was examined from the point of view of various structural materials. It may be stated that most of the significant achievements in the last decade in Japan were culminated in this recent publication.

In this report, the author first reviews various research activities made in Japan since 1968, which in the author's opinion lead to the proposed earthquake load. Second, one of the proposals, called the "First Proposal", is introduced. Finally, a study made by the Reinforced Concrete Committee leading to a modification of the First Proposal is introduced, which suggests a possible and desirable direction for the future earthquake resistant design method for the low-rise reinforced concrete building structures.

HISTORICAL SIGNIFICANCE OF TOKACHIOKI EARTHQUAKE

The proposed "Earthquake Load", which will be explained in detail in the later part of this report, is epoch-making in the history of Japanese earthquake resistant design. In the author's opinion, however, it stemmed out of the experience of Tokachioki earthquake, especially so far as it is related to low-rise reinforced concrete construction.

The Tokachioki earthquake struck northeastern part of Japan on May 16, 1968. From the earthquake engineering point of view this earthquake was characteristic, in the first place, in that it gave Japanese engineers the first recorded strong motion accelerograms of a destructive ground motion in Japan. It was characteristic, in the second place, in that it gave Japanese reinforced concrete buildings the first and the greatest ordeal ever since the beginning of the use of design seismic coefficient in 1924. Many reinforced concrete buildings, especially school buildings, suffered severe damages.

The great Kanto earthquake in 1923 may be regarded as a prehistoric event, because the design seismic coefficient, originally proposed by Professor R. Sano of the University of Tokyo in 1916 [2], was incorporated into the Building Code only in the following year of 1924. The basic requirements for earthquake resistance have remained unchanged since then. Although the standard value of seismic coefficient, 0.1 in 1924 Code, was increased to 0.2 in 1950, this change was accompanied by a corresponding increase in the allowable stresses.

In the course of time several destructive earthquakes hit Japanese reinforced concrete buildings. In 1948, a six-storied department store in Fukui collapsed completely. The building had been subjected to air raid fire in 1945, and it was believed that the collapse was caused by the weakened concrete in the first story columns. In 1964, many reinforced concrete buildings in Niigata suffered uneven settlement or complete overturning due to the liquefaction of saturated sand. Because of the affecting factors, these earthquake damages did not arouse criticism on the earthquake resistant design method itself.

Around 1960 the reinforced concrete construction spread out rapidly to rural parts of Japan. School was the typical example. Partly defrayed by the National Treasury, new school buildings of two- or three-storied reinforced concrete construction were built all over the country. In 1968 when the Tokachioki earthquake hit the northeastern part of Japan, presumably more than 200 modern reinforced concrete buildings had been constructed in the affected area, of which about one quarter were school buildings. All of them had been designed according to the prevalent design method based on the Building Standard Law and Standards issued from the Architectural Institute of Japan. In consequence, medium or heavy damages occurred to approximately 15 percent of these buildings. Counting schools only, it was found that similar or even heavier damage occurred to about 25 percent of existing buildings in the area. Thus the earthquake destroyed the naive belief of citizens that reinforced concrete was an "eternal", earthquake- and fireproof, construction.

More recently, Oita earthquake in 1975 offered lessons to the structural design of Japanese reinforced concrete buildings [3]. However, its significance in the history of earthquake resistant design was never greater than the Tokachioki earthquake. In fact, the impact given by the Tokachioki earthquake triggered most of the recent efforts towards the improvement of earthquake resistant design method of low-rise reinforced concrete. It is the intent of the author to review these efforts including the "Earthquake Load", and to infer the future trend of earthquake resistant design method.

CAUSES OF SHEAR FAILURE OF COLUMNS AND COUNTERMEASURES

Shear failure of reinforced concrete columns was the most typical failure observed in the buildings, especially in the school buildings, on occasion of the Tokachioki earthquake. Three causes were involved there, excluding the everlasting cause of the construction practices: (1) overestimation of allowable shear capacity, (2) inadequate evaluation of shear force distribution, and (3) underestimation of working shear force. More detailed description of these caused and countermeasures taken against them will follow.

Overestimation of Allowable Shear Capacity

Overestimation of shear capacity in the design was the consequence of too high allowable shear stress, too high evaluation of the effect of web reinforcement, and too large spacing in the minimum requirements for web reinforcement. The Ministry of Construction quickly moved towards the partial amendment of the Execution Order of the Building Standard Law, as the most effective countermeasure. Article No. 77 of the Order, dealing with the minimum requirements for column tie spacing, was revised in 1970. Maximum spacing of 30 cm was drastically reduced, to be 15 cm at the middle portion of columns and 10 cm at the top and bottom portions.

The Committee on the Reinforced Concrete Construction of the Architectural Institute of Japan, which is responsible for the Structural Calculation Standard, also responded quickly. A new design equation for allowable shear capacity was developed principally based on Arakawa's research [4]. The equation was incorporated into the revised Standard of 1971 [5].

Shear Force Distribution

The second cause, inadequate evaluation of shear force distribution, resulted mainly from the presence of nonstructural elements. Partition walls, not accounted for as structural shear walls, would carry a considerable share of the story shear force. Spandrel walls, usually neglected in the design calculation, would shorten and stiffen columns, thereby attract more shear, and at the same time they would increase flexural capacity and render the columns more susceptible to shear failure. Such behavior of spandrel walls has been observed in many places affected by earthquakes.

Countermeasure to this phenomenon is, however, not simple. Proposals to take spandrels into design account, or to isolate spandrels from the frame, have been made. But accompanying technological problems, for example how to evaluate flexural and shear capacity of spandrel walls, or how to detail the isolation (expansion) joints, need to be solved. Furthermore, it is of paramount importance to evaluate correctly the overall dynamic response of buildings with or without spandrel walls. If we "cut" all the spandrel walls of existing buildings out of frames, buildings might become too flexible and too weak to withstand earthquakes. We should realize that spandrels are the double-edged swords.

Underestimation of Shear Forces

The third cause was the underestimation of working shear forces on the members. Shear forces had been traditionally determined directly from the design seismic (horizontal) load, ever since the concept of design seismic coefficient was incorporated into the Building Code in 1924. Similar to the treatment of dead and live loads, working forces in the members were computed for the imposed horizontal load, and members were proportioned in such a way that working forces would be exceeded by allowable forces, based on the allowable stresses. This is completely normal and natural procedure from the point of view of the working stress design.

However in view of the fact that the "actual" seismic load could get much greater than the design seismic load, ductility as well as strength should be insured in the design procedure. Premature shear failure should be avoided even if it would take place at a load level well beyond the design seismic load. To achieve this end, shear forces associated with the flexural yield must be used instead of working shear. This was a drastic change in the design concept for Japanese engineers, and it was three years after Tokachioki earthquake, 1971, when the AIJ Standard for reinforced concrete was revised to incorporate the above-mentioned change in the design concept [5].

PREDICTION OF DYNAMIC RESPONSE OF BUILDINGS

By the time when Tokachioki earthquake occurred in 1968, methods of dynamic response analysis had been well developed as design tools for high-rise buildings. Dynamic response analysis considerably broadened the scope of investigations into causes of earthquake damages. Unlike Kanto earthquake of 1923 or Fukui earthquake of 1948, many research projects were carried out where response analysis was applied in order to examine the dynamic behavior of low-rise reinforced concrete buildings.

Researches After Tokachioki Earthquake

Dynamic analysis has to be based on the nonlinear restoring force characteristics if one wants to examine behaviors up to the failure. Laboratory tests of various reinforced concrete structures -- beams, columns, shear walls, beam-column connections, and frames -- had been made under reversal of loading [6]. These available informations were used in the modeling of reinforced concrete hysteresis. Futhermore, a field test was conducted to actually evaluate the hysteresis and modes of failure [7].

A U.S.-Japan Seminar was held in 1970 at Sendai, to exchange the research informations, mainly to investigate the experience of Tokachioki earthquake. Professor J. Penzien of the University of California and Professor H. Umemura of the University of Tokyo were the coordinators. Proceedings were published and are available from the Japan Earthquake Engineering Promotion Society [7]. The seminar turned out to be quite successful, and another seminar of similar character was organized in 1973 at Berkeley, California, this time mainly to examine the consequence of San Fernando earthquake. Professor B. Bresler of the University of California and Professor K. Kubo of the University of Tokyo were the coordinators.

Stimulated by these successful seminars, a cooperative research project on earthquake engineering was undertaken in 1973 - 1975 between the U. S. and Japan. Several researchers were exchanged, and the results were presented at the Review Meeting, held in 1975 at Honolulu, Hawaii. Proceedings were published and are available from the same source [3]. Professors J. Penzien and H. Umemura were coordinators of the cooperative research program.

One of the main themes in the international cooperation was focused on the prediction of dynamic behavior of existing structures [8]. This was made in parallel with other research projects within Japan. Directly following the investigations into causes of earthquake damage of individual buildings, two attempts were started on the basis of successful experience to apply response analysis to low-rise reinforced concrete buildings. The first was the assessment of seismic safety of existing buildings [9,10]. The second was the trial design of buildings which might be judged safe in the light of dynamic response [11,12,13]. Both of them required the prediction of dynamic behavior by some means of response analysis -- rigorous or approximate. Through these investigations criteria for seismic safety of frame buildings and shear wall buildings were established separately, in terms of required strength and associated ductility.

Summarizing all these efforts since the Tokachioki earthquake, it may be stated that the dynamic response of buildings during earthquakes, including the possibility of failure, could be estimated, provided that a mathematical model was set up which would appropriately represent the nonlinear restoring force characteristics. Also it was found that the lateral strength of the buildings governed most directly the magnitude of response deformation.

Variation in Lateral Strength

As an important by-product of these investigations, it was pointed out that the lateral strength of buildings would vary tremendously even though they had been designed for the same seismic coefficient by the same design procedure. The allowable stress of reinforcing steel for seismic loading was increased to yield point in 1950, by which it was intended that ultimate flexural capacity would reflect into the design. However, some buildings would be four or five times as strong as what the design seismic coefficient implied, while others would be only 20 or 30 percent stronger.

Many reasons could be pointed out. Followings are the reasons to inflate the flexural strength of framing girders and columns.

a) Reinforcement may be dictated by permanent loading.
b) Steel area is always rounded up.
c) Because of the working stress equations, members would have excessive strength when concrete stress governs.
d) According to minimum requirements, some bars are always provided where they are not required by calculation.
e) Column reinforcement may be provided for the unfavorable axial force combination.
f) Members of similar function may be unified to the one with largest amount of reinforcement.
g) Bars may be rearranged in the practice for easier placement.
h) Bars usually neglected in the calculation will cooperate; for example floor slab bars to the girder strength, and column bars in one direction to the strength in the other direction.
i) Actual steel yield point will always be greater than the specified minimum.

Secondly the flexural strength of frames may increase due to the following.

j) When design forces and moments are taken at center lines of members, the distance from the center line to the critical section will contribute to the increase of strength.

k) This is particularly significant when spandrel walls are present, which have not been taken into structural design consideration. It was inferred that many reinforced concrete buildings survived the Tokachioki earthquake owing to the increased strength due to the presence of spandrel walls [11].

As to shear walls similar factors as above could be pointed out, resulting even greater reserve strength in many cases. Futhermore most buildings have partitions and other "nonstructural" walls which actually contribute tremendously.

The problem here is that the increase of lateral strength is not dependable. It just does not always happen. Depending on the structural planning and proportioning, some buildings may scarcely have excess strength, yet they may lack in ductility. A more rational design method is needed which would provide more uniform seismic safety.

PROPOSAL OF EARTHQUAKE LOAD (FIRST PROPOSAL)

The Vibration Committee of the Architectural Institute of Japan, which is responsible for the development of design seismic loading, announced two Proposals of "Earthquake Load", in 1973. One of them, called the "First Proposal", was originally made by Drs. M. Izumi, M. Watabe, Y. Matsushima and I. Sakamoto of the Building Research Institute. The other was made by Professors T. Kobori and R. Minai of the Kyoto University, commonly called the "Second Proposal".

Both of them were then subjected to the examination by the "Joint Committee on the Earthquake Load", consisting of representatives from various structural committees under the chairmanship of Professor H. Umemura, with the aim of exchanging opinions from the standpoint of structural materials. In January 1975, Professor Umemura transferred the Proposals and written discussions officially to five structural committees, -- Reinforced Concrete, Prestressed Concrete, Composite Steel and Reinforced Concrete, Steel, and Timber -- asking each committee to make practical examples of application of the First Proposal. Each committee worked hard not only to make such examples but also to criticize and modify the Proposal as needed. These works were joined to a volume of book entitled "Earthquake Load and Earthquake Resistance of Building Structures" [1].

In this section of the report, outline of the First Proposal will be introduced. In the next section, works done by the Reinforced Concrete Committee will be outlined, which, in the opinion of the author, follows directly the flow of works since the Tokachioki earthquake, and suggests how the future design method ought to be.

Classification of Buildings

Common to First and Second Proposals, concept of building classification was introduced in which buildings belonging to different division were to be designed for earthquakes by different methods.

Division One is for buildings where structural calculations are not required, such as the one following the already approved standard design.

Division Two is for buildings to be designed in accordance with the current Building Standard Law only, such as low-rise construction restricted by minimum requirements for wall ratio and so on.

Division Three is for general buildings to be designed considering dynamic effect. Proposed Earthquake Loads are intended for this division.

Division Four is for special buildings, with complicated system, new material or new construction method, whose design and analysis are to be examined individually by a board of specialists.

Scope of the First Proposal

Buildings in Division Three whose structural design has been made in accordance with the Building Standard Law or similar ordinances shall be examined for earthquake motions of maximum intensity by the following method. In this sense the First Proposal is intended, not to provide design seismic load, but to provide means of "post-design" examination. Whatever the design parameters or design procedures are, a building is judged to be satisfactory, if it meets this post-design examination. In this sense the First Proposal provides the "performance type" design criteria rather than the "specification type".

Another important consideration is due to the fact that the Proposal is applicable to Division Three. Buildings are to be designed by anonymous structural engineers. The procedure of examination should then be as plain and simple as possible. Sophisticated analytical procedure, such as inelastic time-history analysis, should be avoided.

Velocity Response Value for Examination

Velocity response value for the post-design examination is given by the following formula.

$$V_D = Z\ G\ S\ D\ V_o \qquad (1)$$

where

V_D : velocity response value for examination.
Z : coefficient for zoning (1.0 - 0.8).
G : coefficient for soil classification as specified in Table 1.
S : spectral value determined by the ratio of natural period in each mode of building T and critical period of ground T_c as in Fig. 1, where T_c is specified in Table 1.
D : coefficient for damping characteristics of building as specified in Table 2.
V_o : standard value of velocity spectrum, taken to be 85 cm/sec.

Values of V_D for $Z = D = 1$ is shown in Fig.2. Figure 3 shows the corresponding acceleration spectra. Although not explicitly stated in the Proposal, it is inferred that the spectra in Figs. 2 and 3 correspond to earthquake motions with maximum ground acceleration of about 0.3 g.

Table 1 Value of T_c and G

Soil Classification	T_c (sec)	G
Class I	0.3	1.0
Class II	0.5	1.2
Class III	0.8	1.5
Class IV	1.2	2.0

Table 2 Value of D

Construction Classification	D
Steel	1.0
RC, PC frame	0.8
RC, PC wall	0.8
SRC	0.8

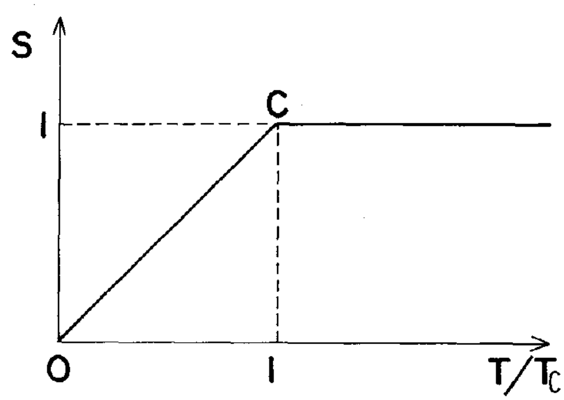


Fig. 1 S vs T/T_c

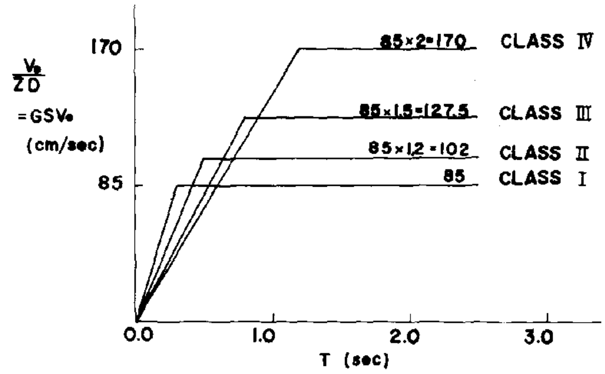


Fig. 2 Velocity Spectra

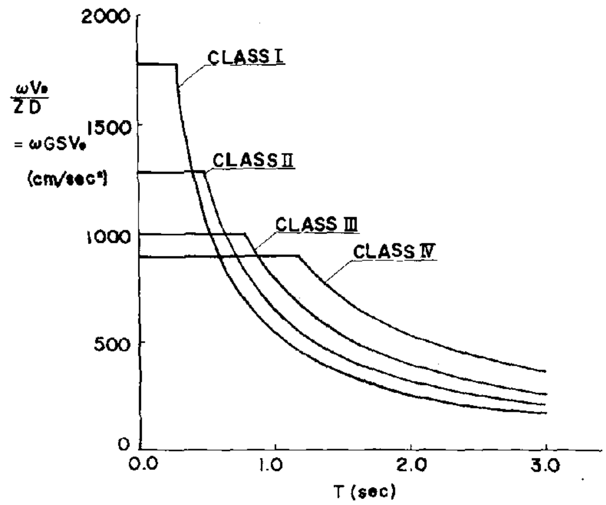


Fig. 3 Acceleration Spectra

Natural Periods and Modes of Building

Natural periods and modes are computed taking stiffness of frames and walls into account. If large amount of sway and rocking at the base of building are expected their effect may be accounted for. However the Proposal does not encourage this consideration, as the method to appropriately evaluate the effect of sway and rocking is not yet established among the Japanese engineers.

Elasto-Plastic Response Displacement and Ductility

Relative displacement and ductility of each story considering elasto-plastic response shall be calculated by the following equations.

$$\delta_r = \mu_r \, \delta_{re} \tag{2}$$

$$\begin{aligned} \mu_r &= \frac{1}{2} \left\{ \left(\frac{Q_{rD}}{Q_{ru}} \right)^2 + 1 \right\} && (Q_{rD} \geqq Q_{ru}) \\ \mu_r &= Q_{rD}/Q_{ru} && (Q_{rD} < Q_{ru}) \end{aligned} \tag{3}$$

where

δ_r : elasto-plastic response relative displacement of r-th story
δ_{re} : elastic limit relative displacement of r-th story
μ_r : ductility factor of r-th story
Q_{ru} : ultimate lateral capacity of r-th story
Q_{rD} : elastic response shear force of r-th story

The elastic response shear force Q_{rD} shall be calculated by the following equation.

$$Q_{rD} = \sqrt{\sum_{i=1}^{k} \left(\sum_{j=r}^{n} m_j \, \beta_i \, u_{ji} \, \omega_i \, V_D \right)^2} \tag{4}$$

where

m_j : mass of j-th floor
β_i : participation factor for i-th mode
u_{ji} : natural mode shape (eigen vector) for i-th mode at j-th floor
ω_i : natural circular frequency of i-th mode
k : maximum order of modes to be considered
n : number of stories

Equations (2) and (3) are derived based on the following two assumptions. First, nonlinear response displacement of a single-degree-of-freedom system is related to the linear response as in Fig. 4, originally proposed by Newmark and others. Implicitly assumed here is that the restoring force characteristics of the building can be idealized into perfectly elasto-plastic hysteresis. Second, above relation is applicable to each story in a multi-story building. This postulates that each story yields simultaneously and deforms to approximately same ductility factor, or at least this

Table 3 Importance Factor

Use of Buildings	I_r		Note
	Lowest story	Uppermost story	
Broadcasting Stations Hospitals	2.0	2.0	Linearly interpolate for intermediate stories
Telephone Exchanges Fire Stations	1.8	1.5	
Government Buildings School Buildings	1.6	1.3	
Others	1.3	1.0	

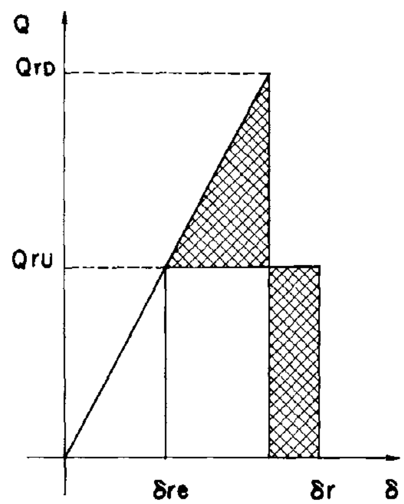


Fig. 4 Linear and Nonlinear Systems

condition is not violated on a large scale.

Equation (4) is so-called modal superposition by root-sum-square law. Since such complicated buildings as might vibrate in torsional modes are excluded out of the scope, maximum order of modes to be considered, k, may be taken to be 3.

Acceptance Criteria for Earthquake Resistance

Buildings are judged to be acceptable when all the following relations are satisfied.

$$\mu_r \leqq \mu_a/I_r \tag{5}$$

$$\delta_r/H_r \leqq 1/125 \tag{6}$$

$$\sum_{r=1}^{n} \delta_r / \sum_{r=1}^{n} H_r \leqq 1/150 \tag{7}$$

where
μ_a : allowable ductility factor determined for each type of construction
I_r : importance factor as specified in Table 3
H_r : height of r-th stroy

Equation (5) requires the response ductility of each story to remain within the allowable limit with a safety margin which is dependent on the importance of the story. Allowable ductility factor μ_a of 5 for steel and SRC, 3.5 for RC frame, 2.0 for RC wall were once proposed, but their final dicision was left to each Structural Committee.

Equations (6) and (7) require the response deformation in terms of translation angle to remain within the prescribed limit. The limiting values are subject to further discussions.

In case the building fails to satisfy these criteria, structural design must be modified. In general, greater strength will be provided to the structure. However, it is permissible to improve detailing so that greater allowable ductility is available.

Examination for overturning and appendages such as penthouse must also be made. Provisions for these items will be added in the future.

"FIRST PROPOSAL" ADAPTED TO REINFORCED CONCRETE

The Reinforced Concrete Committee of the Architectural Institute of Japan, after completion of the work for the revised Building Standard in 1971, was reorganized and started survey of literatures for the next phase revision -- adoption of limit state concept in the design. Particularly important and difficult here was how to define the seismic limit state.

The proposal of the Earthquake Load was made at this time. The principle of the proposal was accepted quite favorably by the Committee, which had been groping for the limit state concept in the earthquake-resistant design.

The approach of the First Proposal was to examine the earthquake resistance by evaluating the ultimate lateral capacity after the proportioning of members. This approach was deemed desirable for the purpose of the Committee because of the following reasons.

First, the greatest shortcoming of the current design method, lack of uniformity in the ultimate lateral capacity, can be most easily overcome by actually evaluating the ultimate lateral capacity. The current design method allows much freedom in the structural planning, design calculation and reinforcement arrangement, which is good as it is. However because of this freedom the ultimate lateral capacity will inevitably fluctuate, from the one barely in excess of design seismic coefficient to the other several times as strong as required. By adopting the First Proposal in the design procedure, such fluctuation is detected, and may or may not be accepted depending on the relation of required versus available ductility. In this way, more uniform earthquake resistance will be achieved.

Second, the approach of the First Proposal follows, in a sense, the same line as the AIJ Standard for Reinforced Concrete. As stated previously, design for shear of reinforced concrete members had a drastic change in 1971, into the ultimate-strength type procedure. Instead of shear forces associated with the design seismic load, shear forces are calculated from the yield moment of sections with flexural reinforcement already arranged. The calculation of ultimate lateral capacity, an essential step in the First Proposal, may be regarded as an extention of the evaluation of ultimate strength after bar arrangement from member level to the structure level.

The Reinforced Concrete Committee attempted to apply the First Proposal to several example buildings, and to modify it, as needed, to a form more suitable to reinforced concrete. Principally there were three points. (1) Definition of the building classification, Divisions Two, Three and Four, was made for concrete structures. (2) Effective period was defined to adapt Eq. (3) of the Earthquake Load, which was based on the perfectly elasto-plastic hysteresis, to the reinforced concrete hysteresis. (3) The procedure of the Earthquake Load was incorporated into a part of member proportioning process.

Definition of Building Classification

As stated earlier, buildings are classified into four divisions, and the Earthquake Load is intended to apply to the Division Three buildings. Division One is for buildings where structural calculations are not required, such as the one following the approved standard design, and no further definition is needed. Hence only Divisions Two and Four are discussed here.

Division Two -- This class is for buildings to be designed in accordance with the current Building Standard Law only, such as low-rise construction restricted by minimum requirements for wall ratio and so on. Reinforced concrete wall buildings and freme buildings with considerable amount of shear wall have been shown to be quite earthquake-resistant in many earthquakes in the past. Figure 5 shows the relation between earthquake damage observed in the Tokachioki earthquake and structural parameters [11]. Abscissa is the horizontal area of walls in one direction at the first story divided by the total floor area, hereafter called wall ratio. Ordinate is total building weight divided by the sum of horizontal area of columns and walls. Buildings having wall ratio greater than 35 cm^2/m^2 are always safe, while those having wall ratio less than that can be made safe only under certain circumstances.

Considering this fact and recent researches into the earthquake-resistant design [3,8,11,12,13] the Committee approved the tentative proposal made by Professor T. Okada of the University of Tokyo to define the Division Two buildings as those having wall ratio greater than 30 cm^2/m^2.

$$a_w = A_w / \sum A_f > 30 \ cm^2/m^2 \tag{8}$$

where

- a_w : wall ratio (cm^2/m^2)
- A_w : horizontal area of walls in one direction at the first story (cm^2)
- $\sum A_f$: total floor area of the building (m^2)

There have also been more sophisticated proposals [13], but the above-mentioned one would be the most practical and yet reasonably effective.

Division Four -- Buildings in this Division are described as those having "complicated system, new material or new construction method". Buildings with predominant torsional vibration, and those with discontinuous vertical distribution of strength or stiffness were recognized by the Committee to be typical of Division Four buildings. Through the survey of buildings subjected to the Tokachioki earthquake, following was proposed by Professor A. Shibata of the Tohoku University as the practical limit for this Division.

$$e' = e/i > 0.3 \tag{9}$$

$$j' = \sqrt{K_T/K_S}\ /i < 1.0 \tag{10}$$

where

- e' : eccentricity ratio
- e : eccentricity length between center of gravity and center of translational stiffness
- i : radius of gyration
- j' : stiffness radius ratio
- K_T : torsional stiffness
- K_S : translational stiffness

As to the buildings with sudden change in strength or stiffness, such as piloti or soft-first-story building, it has been shown that the ductility of multi-degrees-of-freedom shear model can be approximately evaluated from the linear response as follows (Fig. 6) [14].

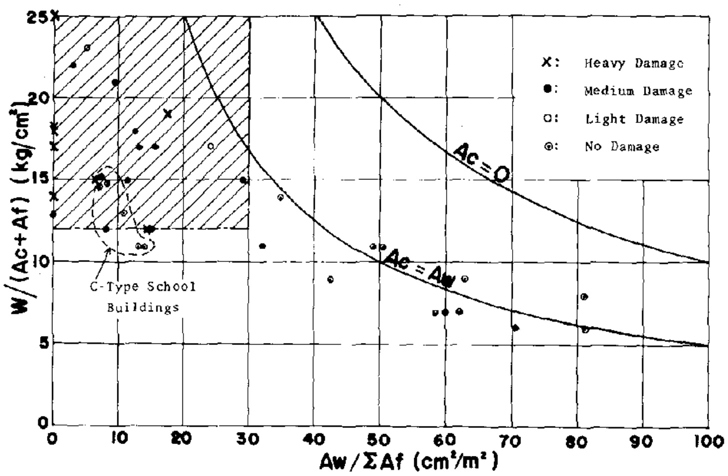


Fig. 5 Wall Ratio and Earthquake Damage

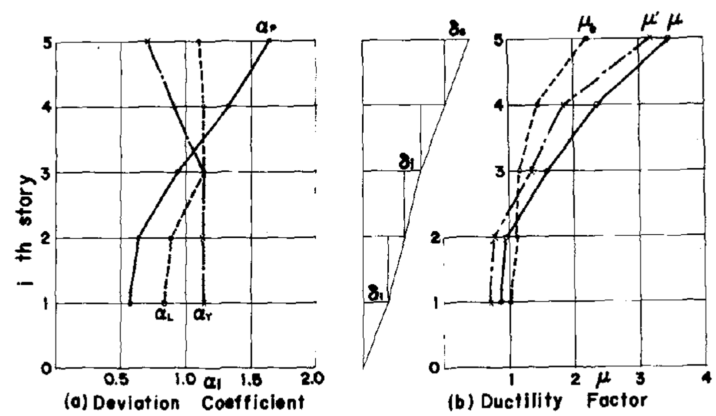


Fig. 6 Nonlinear and Linear Responses

$$\mu_i \simeq \mu_i' = \frac{\alpha_{Li}}{\alpha_{Yi}} \mu_{ei} \tag{11}$$

where

μ_i : response ductility factor (of i-th story)

$$\mu_i = \delta_{pi}/\delta_{Yi} \tag{12}$$

μ_i' : approximate value of μ_i

μ_{ei} : apparent ductility

$$\mu_{ei} = \delta_{Li}/\delta_{Yi} \tag{13}$$

α_{Li} : deviation of linear response story-displacement

$$\alpha_{Li} = n\ \delta_{Li}/\sum_{i=1}^{n} \delta_{Li} \tag{14}$$

α_{Yi} : deviation of yield story-displacement

$$\alpha_{Yi} = n\ \delta_{Yi}/\sum_{i=1}^{n} \delta_{Yi} \tag{15}$$

δ_{Li} : linear response story-displacement

δ_{pi} : nonlinear response story-displacement

δ_{Yi} : yield story-displacement

n : number of stories

Effective Period for D-Tri Response Estimation

The restoring force characteristic of reinforced concrete structures failing primarily in flexure can be idealized into so-called degrading trilinear (D-Tri) model, shown in Fig. 7 [3,15]. As an example a family of displacement response spectra for various yield strength is shown in Fig. 8. Cracking and yield strengths are expressed in terms of seismic coefficient, and the maximum acceleration of input ground motion is normalized to 1.0 g. Yield stiffness ratio α_y is taken to be 0.5. As seen here the displacement in the long period range is close to the linear response, but it increases drastically from linear response value in the short period range and for low yield strength.

For the single-degree-of-freedom system having yield strength of k_y in terms of seismic coefficient and subjected to earthquake with maximum acceleration coefficient of k_g, nonlinear response displacement can be approximated by

$$\delta = \frac{1}{2} \{ (\frac{k_e(T)}{k_y})^2 + 1 \}\ \delta_y(T) \tag{16}$$

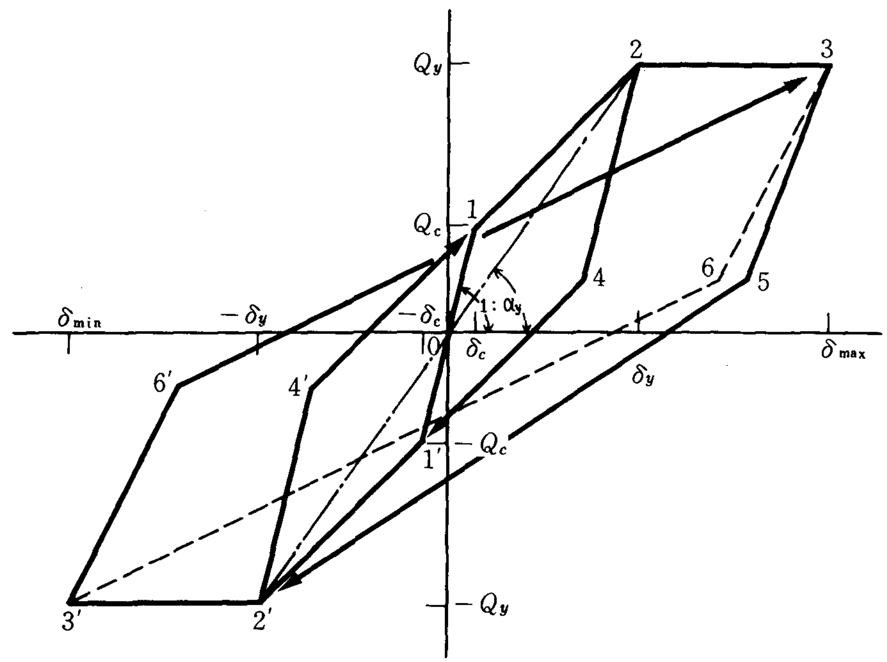


Fig. 7 Degrading Trilinear Model

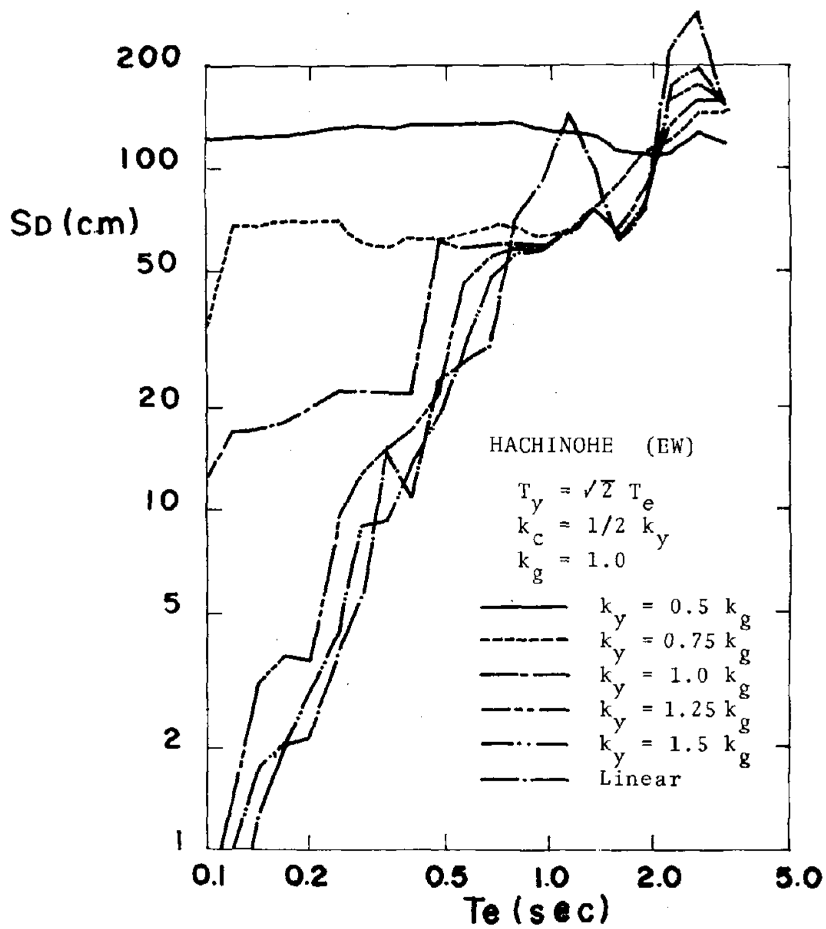


Fig. 8 D-Tri Response to Hachinohe Record
of Tokachioki Earthquake

where

δ : nonlinear response displacement
$k_e(T)$: linear response shear coefficient for period T
k_y : yield shear coefficient
$\delta_y(T)$: yield displacement for period T

and the period T is selected by the following rule.

$$
\begin{aligned}
T &= T_e && \text{if} \quad T_e \geqq T_{eq} \\
T &= T_{eq} && \text{if} \quad T_y > T_{eq} > T_e \\
T &= T_y && \text{if} \quad T_y \leqq T_{eq}
\end{aligned}
\tag{17}
$$

where

T_e : period associated with initial (uncracked) stiffness
T_y : period associated with yield stiffness ($T_y = T_e/\sqrt{\alpha_y}$)
T_{eq} : equivalent period from Eq. (18)

$$T_{eq} = 1.5 - k_y/k_g \tag{18}$$

k_g : maximum acceleration coefficient

The above relations are shown in Fig. 9. Equations (16), (17) and (18) select an equivalent period T_{eq}, which determines a constant nonlinear response for intermediate range of period. For longer period range, nonlinear displacement is evaluated by Eq. (16) using "elastic" period T_e, which is usually on the safe side because in this period range nonlinear displacement is closer to linear displacement. For shorter period range, nonlinear displacement is found using "yield" period T_y, which is a good approximation for k_y values not less than k_g. In this period range Eq. (16) underestimates the response if k_y is less than k_g, but the ductility factor in this case is altogether so large that it is anyway out of practical significance.

The condition in Eqs. (17) and (18) is equivalent to the following, equating k_g to 0.3.

$$
\begin{aligned}
T &= T_e && \text{if} \quad k_y \geqq k_{y1} \\
T &= T_{eq} && \text{if} \quad k_{y1} > k_y > k_{y2} \\
T &= T_e/\sqrt{\alpha_y} && \text{if} \quad k_y \leqq k_{y2}
\end{aligned}
\tag{19}
$$

where

$$k_{y1} = 0.3\ (1.5 - T_e) \tag{20}$$

$$k_{y2} = 0.3\ (1.5 - T_e/\sqrt{\alpha_y}) \tag{21}$$

$$T_{eq} = 1.5 - k_y/0.3 \tag{22}$$

The rule of Eq. (19) is illustrated in Fig. 10. As seen here this rule specifies that an effective period be used in case both elastic period and

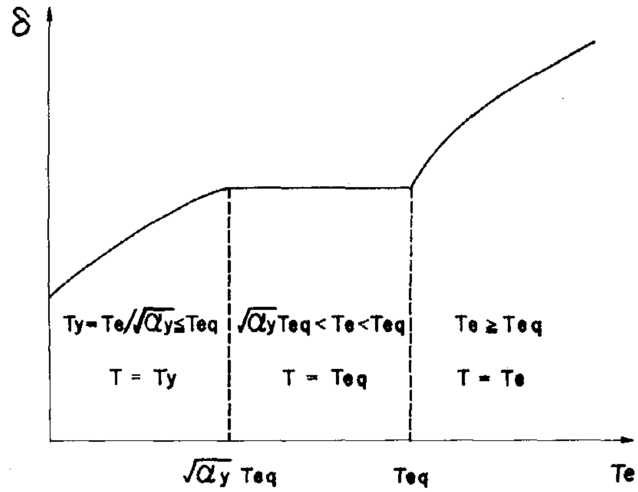


Fig. 9 Rule for D-Tri Response Evaluation (1)

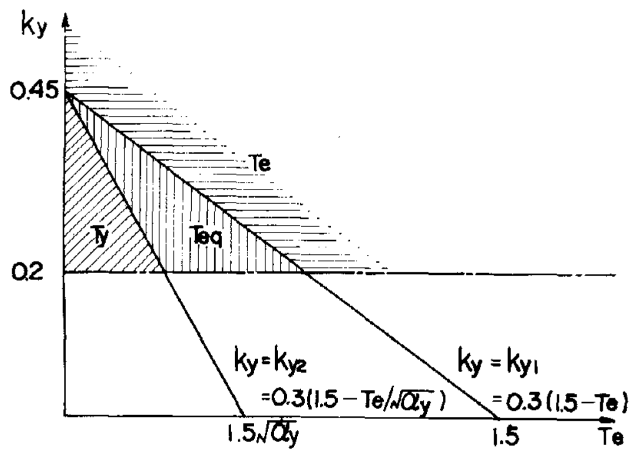


Fig. 10 Rule for D-Tri Response Evaluation (2)

yield shear coefficient are small.

For the class II soil, zoning coefficient Z = 1, and for reinforced concrete D = 0.8, we have from Eq. (1)

$$V_D = \begin{array}{ll} 163\ T \ \text{cm/sec} & (T \leq 0.5 \text{ sec.}) \\ 81.6 \ \text{cm/sec} & (T \geqq 0.5 \text{ sec.}) \end{array} \quad (23)$$

Assuming that the maximum ground acceleration associated with the response of Eq. (23) is 0.3 g, we obtain linear and nonlinear response displacement spectra for 1.0 g earthquake as shown in Fig. 11. Yield stiffness ratio α_y of 0.5 was used here. Figure 11 is directly comparable to Fig. 8. When such comparison was made for several earthquake records and for different parameters as shown in Fig. 12, it was concluded that the above-mentioned effective period may be used for the evaluation of D-Tri response in the range of ductility factor up to about 5. Figures 13, 14, 15 and 16 are the response ductility and displacement spectra for four classes of soil calculated by this method, where ductility factor is taken from the yield displacement.

Modified "First Proposal"

Based on the foregoing study, following modification to the First Proposal was proposed by the author.

a. Calculate natural periods and participation functions of each mode based on the elastic (uncracked) stiffness.
b. Evaluate the yield stiffness ratio α_y either by calculation or guess work. Recommended is a value of 0.5.
c. Calculate the ultimate lateral capacity of each story, and obtain the yield shear coefficient from the following equation.

$$k_y = \frac{\sum_{i=1}^{n} Q_{ru} h_r}{\sum_{i=1}^{n} (W_r \sum_{j=1}^{r} h_j)} \quad (24)$$

where
Q_{ru} : ultimate lateral capacity of r-th story
h_r : story height of r-th story
W_r : weight of r-th floor

Equation (24) was derived from the equation of motion in the plastic flow assuming that the mode shape was an inverted triangle.

d. Obtain the effective natural period of the first mode from Eqs. (19), (20), (21) and (22) where T_e is the elastic natural period of the first mode. If it is different from the elastic period, modify all the higher mode periods by the same ratio as that of the first mode.

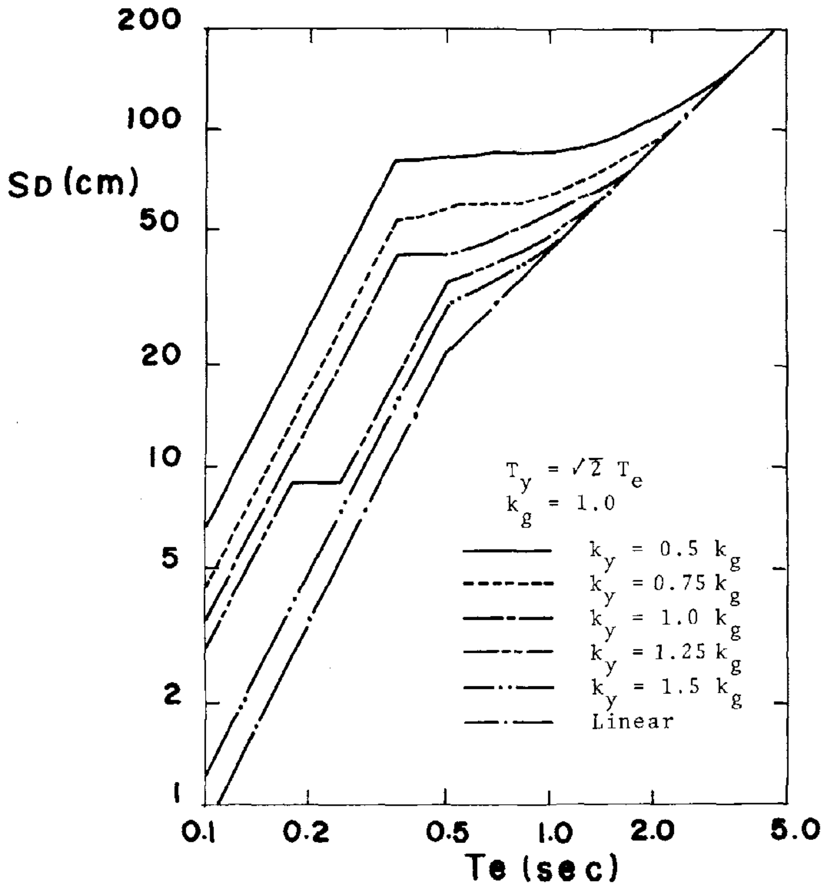
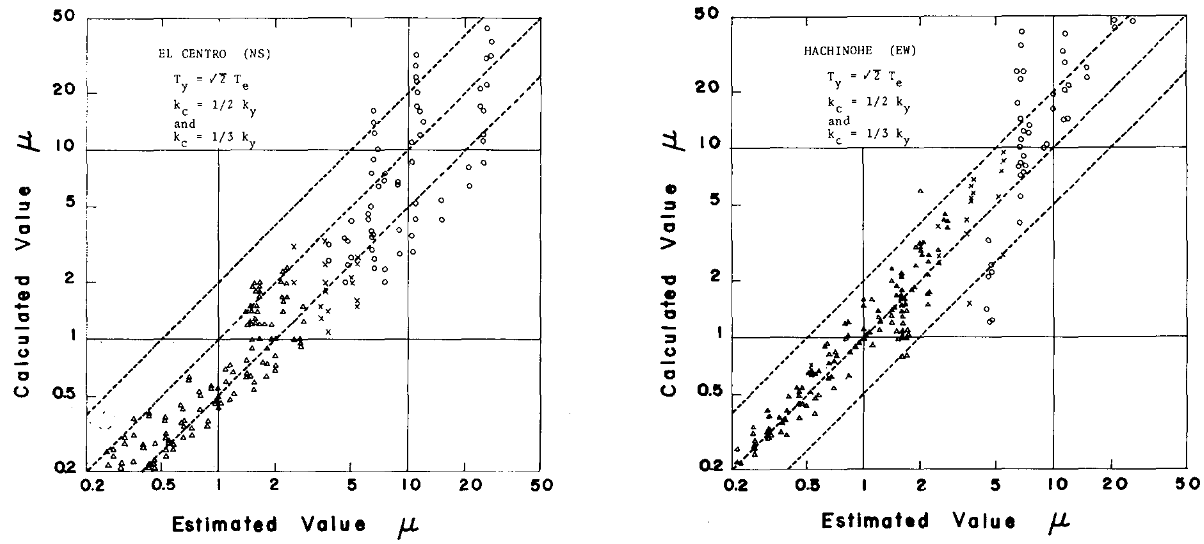


Fig. 11 Estimated D-Tri Response



(a) El Centro - 1940

(b) Hachinohe - 1968

Fig. 12 Comparison of Calculated vs Estimated Values

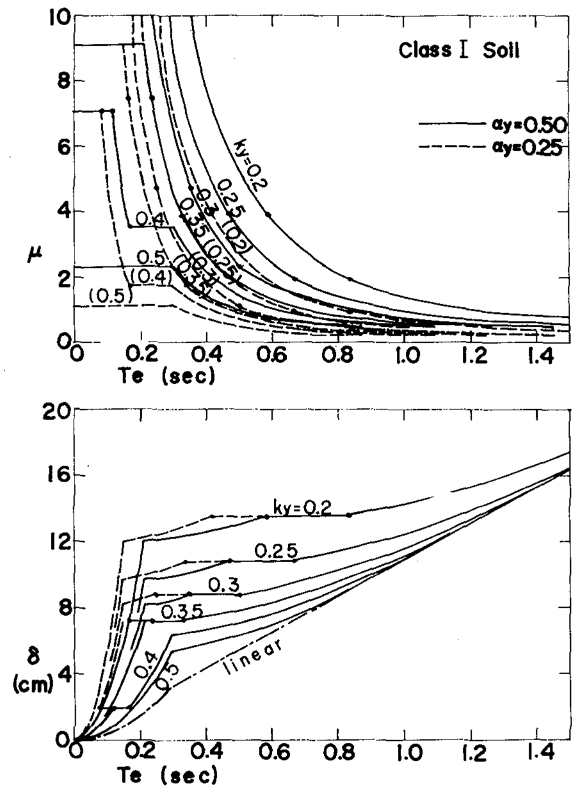


Fig. 13 Response for Class I Soil

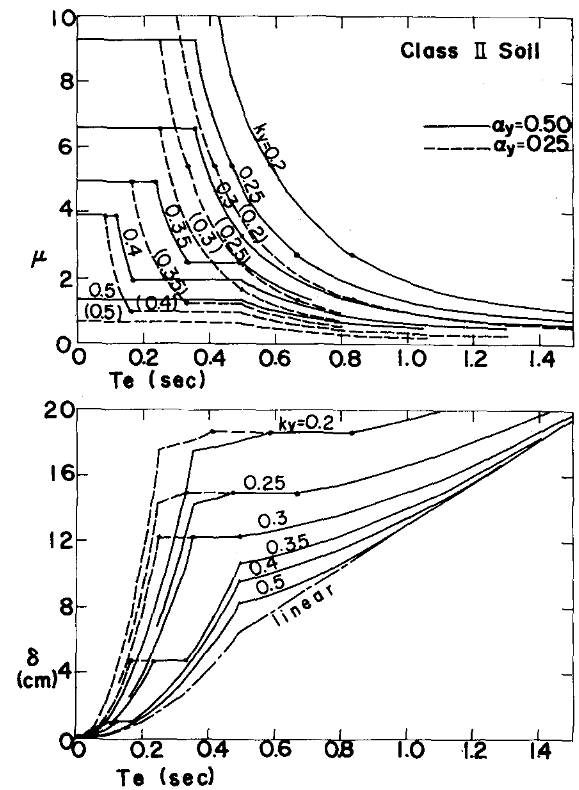


Fig. 14 Response for Class II Soil

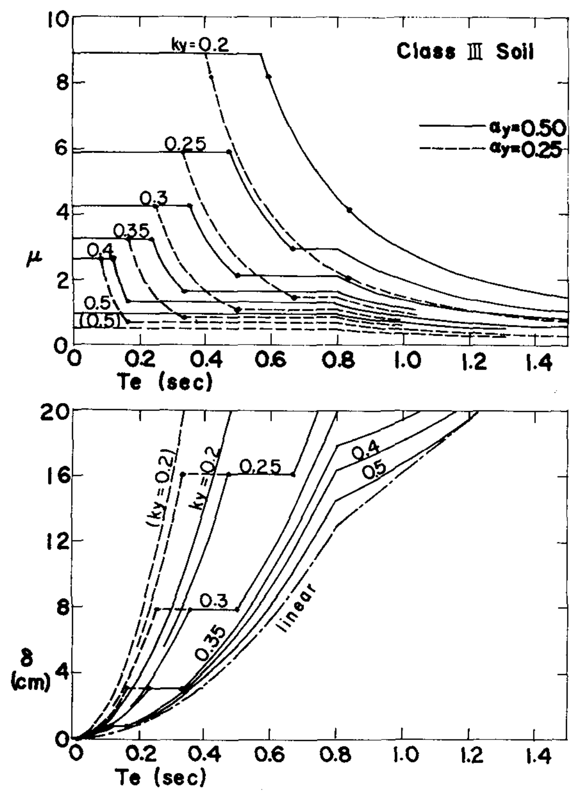


Fig. 15 Response for Class III Soil

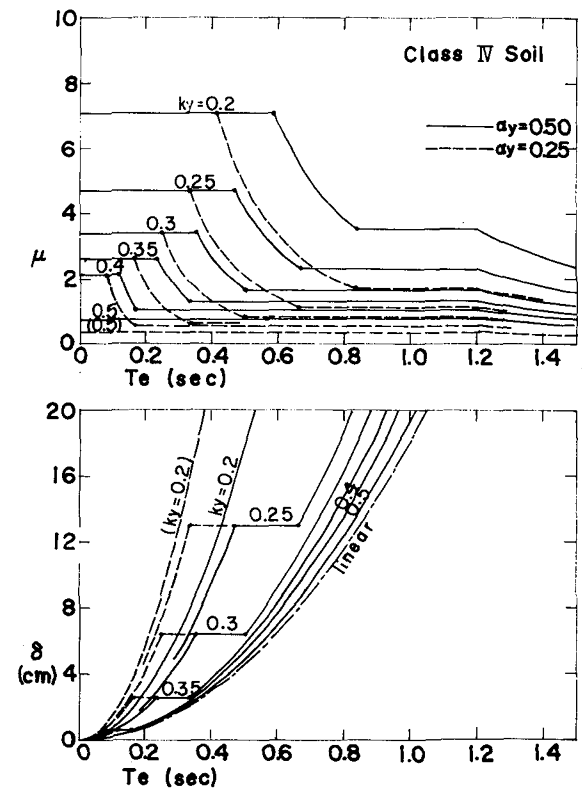


Fig. 16 Response for Class IV Soil

e. Find the linear response shear force in each story from Eqs. (1) and (4), using the periods modified as above.
f. Find the ductility factor in each story, defined for the yield displacement from the following expressions.

If $k_y \geqq k_{y1}$

$$\mu_r = \frac{\alpha_y}{2}\{(\frac{Q_{rD}}{Q_{ru}})^2 + 1\} \qquad (Q_{rD} \geqq Q_{ru})$$

$$\mu_r = \alpha_y Q_{rD}/Q_{ru} \qquad (Q_{rD} < Q_{ru}) \qquad (25)$$

If $k_{y1} > k_y > k_{y2}$

$$\mu_r = (\frac{T_{eq}}{T_1})^2 \frac{\alpha_y}{2}\{(\frac{Q_{rD}}{Q_{ru}})^2 + 1\} \qquad (26)$$

If $k_y \leqq k_{y2}$

$$\mu_r = \frac{1}{2}\{(\frac{Q_{rD}}{Q_{ru}})^2 + 1\} \qquad (27)$$

where
μ_r : ductility factor of r-th story defined for yield displacement
α_y : yield stiffness ratio
Q_{rD} : *linear response* shear force of r-th story
Q_{ru} : ultimate lateral capacity of r-th story
T_1 : elastic natural period of first mode
k_y, k_{y1}, k_{y2}, T_{eq} : refer to Eqs. (24), (20), (21), (22), use T_1 for T_e in Eqs. (20) and (21)

g. Calculate nonlinear response story displacement from the following equation.

$$\delta_r = \mu_r \, \delta_{ry} \qquad (28)$$

where
δ_r : nonlinear response story displacement of r-th story
δ_{ry} : yield story displacement of r-th story

Criteria for Earthquake Resistance

The Reinforced Concrete Committee examined the acceptance criteria of the original "First Proposal", and temporarily concluded as follows.

(1) Although the adoption of importance factor is plausible, the quantitative definition is still very difficult. Hence it will not be further discussed within the Committee at least for the time being.

(2) The principal criteria should be in terms of story slope (story

displacement divided by story height). Ductility factor has been used more frequently here and abroad. However the story ductility factor to be found by the "First Proposal" is unrelated to the member ductility factor. Futhermore the story ductility factor becomes ambiguous due to crude evaluation of stiffness reduction factor, while the inelastic response story displacement, or story slope, is more definitely evaluated. Hence the story slope is used as the principal criteria, and the total slope (total displacement at the top of the building divided by total height) and ductility factor are referred to only as auxiliary criteria.

(3) The permissible limits are set as follows based on the engineering judgment considering available experiences and experimental data.

Story slope : $\delta_r / H_r \leqq 1/100$ (29)

Total slope : $\sum_{r=1}^{n} \delta_r \,/\, \sum_{r=1}^{n} H_r \leqq 1/120$ (30)

Ductility : $\mu_r \leqq 3.5$ (31)

(4) A new concept of "ductility class" is introduced. According to the calculated story slope, columns and girders belonging to each story are classified into three classes as shown in Table 4, and they must be designed for shear by appropriate methods to ensure deformability associated with each class.

The detail of design methods for shear will be developed in future. There will be differences among classes of the following items: evaluation of seismic shear force in the members, shear capacity equation, limiting value of axial compression, limiting value of web reinforcement, limiting value of shear span ratio, criteria for development length, and confinement for bond splitting.

FUTURE TREND IN EARTHQUAKE-RESISTANT DESIGN

The Reinforced Concrete Committee resumed its study towards the limit state design code when the works for the "Earthquake Load" was completed in March, 1977. Fundamental scheme for the future code is not yet laid out. However, the author views personally that the modification made to the "First Proposal of the Earthquake Load" will constitute the skeleton of the future code by itself.

Figure 17 shows the flow diagram of design procedure proposed by the Reinforced Concrete Committee in the modification of the "First Proposal". Traditional design follows the left branch directly. The right branch is the new addition, which is quite simple and easy for most structural engineers.

It is necessary for the Committee to examine various problems in order to develop details of the design procedure, among which are the followings.

a) Define clearly the scope of application of the design flow shown in Fig. 17. In other words define more clearly the Divisions Two, Three and Four.

b) Choose appropriate design seismic coefficient (or load factor to the seismic coefficient in the Building Standard Law) to design most efficiently, avoiding the circulation shown by the dashed arrow in Fig. 17.

c) Select, or develop as needed, ultimate strength equations for flexure and shear in conjunction with the associated deformability, and determine material safety factor or member capacity reduction factor.

d) Provide effective means to calculate ultimate lateral capacity, both for hand and automatic calculations.

e) Determine permissible limits of response deformation and ductility more firmly.

Obviously we would have to start with some of these items left for the "engineering judgment". To the author it seems that the adoption of the scheme shown in Fig. 17 is a natural consequence of all kinds of studies made since the Tokachioki earthquake of 1968. We should switch our design procedure from the conventional specification type design earthquake load to the performance type criteria as in Fig. 17.

Table 4 Ductility Class

Calculated Story Slope R	Ductility Class
$1/150 \leqq R < 1/100$	I
$1/200 \leqq R < 1/150$	II
$R < 1/200$	III

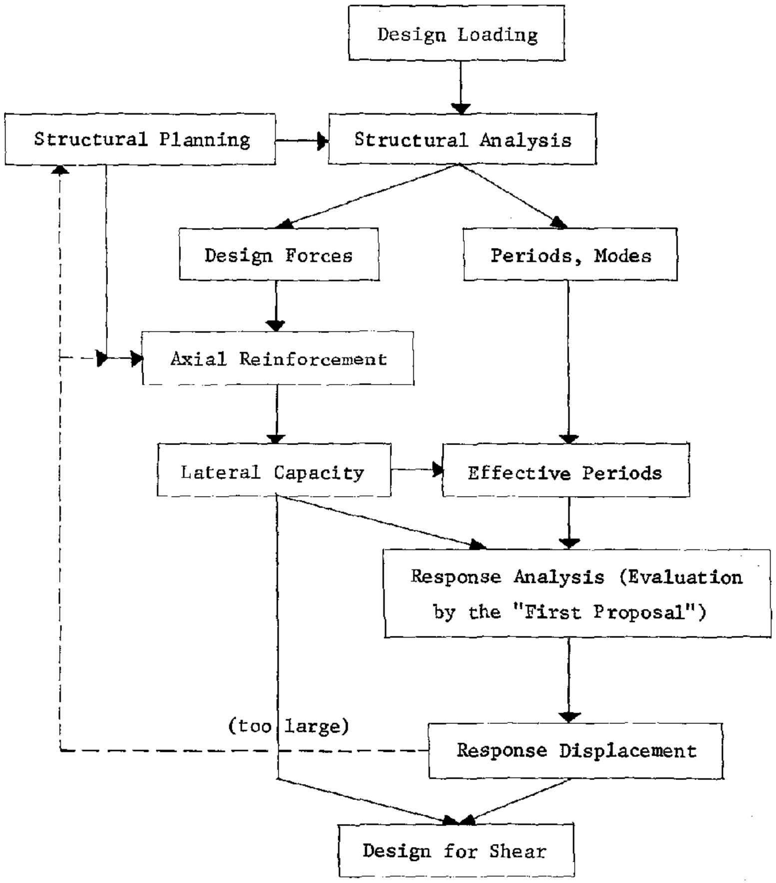


Fig. 17 Flow Chart for Earthquake Resistant Design

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SEISMIC DESIGN REQUIREMENTS IN A MEXICAN 1976 CODE

by

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Introduction

Mexico City contains over one-sixth of the country's population, and its concentration of engineered buildings is much more pronounced. The major part of the city lies within the Federal District. The District's building codes are accordingly the most advanced in Mexico.

The country's and Federal District's first building code containing earthquake-resistant provisions dates from 1942. A macroseism in 1957 prompted the issue of emergency regulations (1), which were beset with the consequences of haste. The compass was corrected in the 1966 code, which was essentially complete years earlier (2). Then came the Federal Electricity Commission norms (3), whose innovations served as basis for the 1976 code. The latter is in large measure also the outcome of research done at the Instituto de Ingeniería of the National University and of interchange of ideas and experiences with the Applied Technology Council of the United States.

The authority responsible for drafting codes and for issuing construction and occupation permits is the Federal District Department. It usually places responsibility for complying with code provisions in hands of the registered engineer or architect that is awarded the construction license. Department engineers check computations and drawings only for some special structures and for those severely damaged by earthquake. Codes often serve more as guidelines than as norms. There is much freedom in design of privately owned buildings while those belonging to government organizations follow their own provisions. The same pattern holds for construction supervision.

The situation is more orderly than might be construed since in cases leading to penal responsibility or legal dispute, having violated code requirements weakens a professional's position.

Many states in the republic are now (1976) in the process of approving new building codes, carefully adopted from that of the Federal District.

Seismicity

Soil amplification, particularly for long periods, is extreme in the soft clay of the Valley of Mexico. In acceleration spectra, prevailing periods of 2 to 2.5 sec are common (4,5) and at one site they have reached 5 sec (Fig. 1).

On hard ground the 100 yr return period ground acceleration and velocity are respectively 0.05 g and 20 cm/sec (8 in/sec), where g = acceleration of gravity. The corresponding values on soft ground are 0.14 g and 81 cm/sec (32 in./sec) (7). These values refer to free field. Considerable reductions may be expected at the base of ordinary buildings owing to foundation rigidity in a horizontal plane (8), foundation depth (9), and sometimes soil-structure interaction (10).

General Requirements

Importance Factor.--Since 1966 the factor that distinguishes between essential facilities and ordinary buildings has become 1.3, much smaller than in previous codes. The lower figure can be justified by considering that the optimum base shear coefficient is nearly proportional to the $1/(r+1)$ power of the expected loss in case of failure (10), where $-r$ is the exponent in

$$\lambda(S) = \alpha(S/S_m)^{-r} \qquad (1)$$

in which $\lambda(S)$ = exceedance rate of structural response S; α and r = parameters depending on regional seismicity, structural properties, and nature of response being considered; and S_m = smallest value of S for which we have complete information. On hard ground in Mexico City r depends on natural period and ranges between 2.39 for maximum ground velocity and 2.70 for maximum ground acceleration (11). On soft ground the range is 2.60 to 3.75 (7). Accordingly an importance factor of 1.3 implies an expected loss of 2.4 to 3.5 times greater for essential facilities than for ordinary permanent structures, other things being equal. These ratios seem reasonable. Yet a thorough study of the matter would be much in order.

Fence walls whose height does not exceed 2.5 m (8 ft) and temporary warehouses are the only structures that need not be designed to resist earthquakes.

Microzoning.--The four zones are, I hard ground, II transition, III soft soil, and IV insufficiently explored. Reclassification of a site in zone IV into another zone is to be based on local soil exploration. However, if information is insufficient to define period T_2 in the acceleration spectrum (see below), this period should be assumed equal to 5 sec.

Base shear coefficients C for the different zones, associated with the flat portion of acceleration spectra and a ductility factor of 1, appear in Table 1. Recent data (12) indicate that it may be advisable to reclassify part of zone I as II (areas where lava flows are underlain by compressible clay) and lower the value of C for zone I. Results would agree closely with those of design optimization. It is not seriously objectionable to preserve values in Table 1 until a change is better substantiated, for C is already low for zone I and consideration of strong nearby earthquakes may substantiate the table.

Load Factors.--Load factors for gravity forces are 1.4 in ordinary buildings and 1.5 in special structures under uniform live load. These factors are reduced to 1.1 in design under the most unfavorable live-load distribution (checkerboard loading).

In design against wind or earthquake the load factor is also specified as 1.1.

Cross-sectional dimensions of reinforced concrete members smaller than 20 cm (8 in) are assumed to be reduced by 2 cm (0.8 in) except when specially controlled. The reduction does not apply to effective depth for positive moment in beams.

Code load factors, stress-reduction factors, accidential eccentricities in columns, and reductions in cross-sectional dimensions of reinforced concrete members were computed using a second-moment approach and then they were rounded off.

Design Spectra.--Figure 2 depicts the design spectra specified for computation of deformations in structures resting on each of the first three zones. See also Table 1. Shapes are based on spectra of motions recorded since December 1959, when the first strong-motion record was obtained. There are, though, two important modifications relative to the actual spectra:

1. The flat portions were widened to take into account uncertainty in spectrum shapes and in natural periods. Originally it was thought didactically appropriate that the design spectral ordinate associated with a computed period T should be taken as the largest ordinate in the range 0.75T to 1.33T. [Asymmetry of these coefficients relative to 1.00 stems from assigning T a lognormal distribution, which is a reasonable assumption (13).] Reactions of practicing engineers indicated, though, that it would be preferable to modify the shapes of design spectra so that, entering with computed periods, one would directly find the spectral ordinates of the original scheme.

2. The descending branch in actual spectra varies approximately as T^{-1} and T^{-2} in zones I and III, respectively. Assuming that intial costs and losses in case of failure were affected in the same proportion as one varied the number of stories of a building and hence its fundamental period, optimum design would lead to a variation of design ordinates proportional to the foregoing powers of T to the power $r/(r+1)$ (14). With the values of r quoted earlier one would arrive at descending branches in design spectra proportional to $T^{-0.73}$ in zone I and $T^{-1.58}$ in zone III. Longer periods are associated with the possibility of unfavorable behavior caused by phenomena not normally considered in analysis, such as concentration of ductility demand, some forms of soil-structure interaction, particularly in the range of nonlinear behavior, and P-δ effects in excess of computed values. Hence the desirability of even slower variation of design spectral ordinates with T and hence exponents -1/2 and -1 for zones I and III, respectively. The exponent for zone II was interpolated between these values.

The design spectra are intended to correspond to a damping ratio of 5%.

In single-degree hysteretic systems whose initial period exceeds T_1, maximum deformations depend little on the shape of the force-deformation curve provided this is symmetrical and initial stiffness is preserved (13). If the force-deformation curve is elastoplastic, forces induced equal those in the equivalent linear systems (having the same parameters as the elastoplastic one for small deformations) divided by the elastoplastic system's ductility factor. When T = 0 accelerations in the system are independent of its force-deformation relation since it follows the ground motion without perceptible deformation. In the range $0 \leq T \leq T_1$ it is reasonable to interpolate linearly between reduction factors at the ends of the interval. Deformations in the range $0 \leq T \leq T_1$

are to be computed from a spectrum whose ordinates are the ductility factors times the reduced spectral accelerations (Fig. 3).

Design of Ductile Multidegree Systems.--The foregoing criteria carry over approximately onto structures with several degrees of freedom but systematically introduce errors on the unsafe side: individual story ductility factors systematically exceed the overall ductility factor (13). The error increases as the structure departs from one having an approximately uniform load factor throughout. One way of recognizing this situation lies in using smaller ductilities in design than would be derived from individual story behavior, making the available (allowable) ductility a decreasing function of heterogeneity of story-shear load factors. This is subsequently made explicit.

Allowable Ductilities.--Ductility factors to be used in design of single-degree systems are conservative approximations to laboratory test results under several dozen cycles of alternating load (13). They depend on structural materials, on properties of structural members and joints, and on structural details. Reductions mentioned in the preceding paragraph have been incorporated into the allowable ductility factors of multistory buildings. Code requirements for different allowable ductility factors are synthesized in Table 2.

Equating reduction factors to ductility factor in the range $T \geq T_1$ assumes symmetric force-deformation curves and strongly hysteretic behavior. According to Ref. 3 the ductility factor to be used in design should be taken equal to the one in Table 2 times $(1 + 10V_1/V_2)/(5 + 6V_1/V_2)$ when the force-deformation curve in one or more stories can be idealized as in Fig. 4, and the ductility factor μ should be replaced with $\sqrt{2\mu-1}$ when the force-deformation curve can be idealized in Figs. 5 or 6.

A typical condition leading to a force-deformation curve as in Fig. 4 is illustrated in Fig. 7. The girder supporting the single-story columns is aided by gravity in resisting roof inertia forces from left to right. Gravity effects decrease the girder's capacity to resist roof inertia forces acting in the opposite sense. Hence the asymmetry of the curve in Fig. 4. In concrete structures, however, reinforcement can be designed in such a way as to counteract this effect, restoring the reduction factor equality with the ductility factor.

Behavior schematized in Fig. 5 is typical of X-braced structures and chimney stacks anchored with long, ductile bolts. Figure 6 represents behavior of prestressed concrete structures failing through tension in the prestressing tendons: departure from linear behavior is mainly due to opening of cracks which close upon removal of lateral load. The thin hysteretic loop essentially reflects energy losses due to friction between tendons and concrete.

Redundancy.--Among the (random) capacities of critical sections in structural members there is no more than partial correlation. The coefficient of variation of the shear capacity of a series of parallel frames having nearly equal columns decreases therefore as the number of columns increases. The effect is less pronounced if one or two columns have high effective relative rigidities and hence take a large share of the story shear. The situation is covered in the code by specifying that the generalized force acting on every shear wall or column that takes up more than 20% of the story generalized force be increased 20%. The first 20% is such that the provision affects buildings on four nominally equal columns. The second 20% is consistent with an analysis taking into account the dependence of the coefficient of variation of structural capacity on the number of nominally equal columns.

The increase in safety of inverted pendulums brought about through this provision is smaller than the one achieved through an increase in base shear coefficient in previous codes. The main reason for this is that effects of rotational inertia are now explicitly to be taken into account, as described subsequently.

Drift Limitations.--The code limits computed drifts to 0.008 in ordinary structures. The limit is apparently high compared with previous code limits just as base shear coefficients are comparatively high. Both situations are due to specifying spectra directly for computation of deformations while spectra for computation of forces are to be derived from the former through a reduction scheme. Previous codes specified the spectra for force computation (which made distinctions based on available ductilities awkward) so they had to compensate for the reduction when comparing computed and allowable drifts. They did this by allowing only reduced, unrealistic drifts.

The question, however, is not so clear-cut. We should limit drift essentially to control serviceability. We should therefore be more interested in shorter return periods than when we are concerned with collapse. We should even adopt different design spectral shapes for design against collapse than for drift limitations. For the sake of simplicity the code specifies a single spectral shape for a given building. It remains to discern whether the simplification does not introduce excessive errors in some range of fundamental periods.

Drifts limited by this code provision are those associated with frame distortion, not with overall flexure, as in essence only the former cause cracking of nonstructural elements. When the latter are tied to the structure in such a way as not to be damaged by structural deformations the 0.008 limit is doubled. It may seem too arbitrary to impose any limitation under these conditions, but, if it were not imposed, excessively flexible buildings would be produced, designed for very small lateral forces.

P-δ Effects.--To save most buildings from revision of P-δ effects it is permissible to ignore these effects when the computed total drift does not exceed 0.008 in any story. When this condition is not fulfillled, computed column moments and story drifts must be divided by $1 - 1.2c/\psi$, where c = story shear divided by weight above and ψ = computed story drift, and equilibrium must be restored at intersections with floor systems. The correction factor is derived in Ref. 15. Division by $1 - 1.2\psi/c$ is equivalent to division by the more usual form $1 - P/P_{cr}$ where P = weight of building above story being considered and P_{cr} = value of P that would make the story buckle. Factor 1.2 corrects for the difference between the deformed shapes of columns under vertical and lateral forces. Alternative second-order methods of analysis are also allowed.

In addition one must amplify the bending moments acting on individual slender columns. The procedure specified is the one in current "ACI Building Code" (16) except that frames having a total drift not exceeding 0.008 may be considered as restricted against sway. This exception is justified because the assumption that a frame is not restricted implies that all columns tend to buckle simultaneously, and this global phenomenon is covered by consideration of P-δ effects with ψ corresponding to the original design spectrum and c based on the spectral ordinates reduced to account for ductility, i.e., to account for nonlinear behavior. Yet it is inconsistent that a very conservative criterion should govern as soon as ψ exceeds 0.008. The matter deserves further scrutiny.

Combined Action of Ground Motion Components.--In Mexico City vertical ground accelerations are usually not significant (13). A satisfactory idealization of ground motions will therefore consider two orthogonal horizontal components acting simultaneously. It is to be assumed that a structure is safe if state vector $\underline{R}$ falls within the safe region in states space, and $\underline{R}$ is to be computed from

$$\underline{R} = \underline{R}_0 \pm \underline{R}_1 \pm 0.3\underline{R}_2 \tag{2}$$

where $\underline{R}_0$ = vector of gravity effects and $\underline{R}_1$ = vector of ith-component effects (17). Equation 2 is a simple approximation to results of a more rigorous analysis. Even this simple expression involves an increase in accountacy over what is required when the matter is ignored. But unless this is done we systematically underdesign certain structural members (typically corner columns) or, if we raise base shear coefficients to make up for neglect of the question, we overdesign other structural members.

Simplified Method.--The majority of engineered houses and buildings have no serious problems of torsion, overturning, drift, P-δ effects, or combined action of different ground-motion components. These structures are usually little sensitive to the design base shear or to the computed capacity of their structural elements. It is desirable to include in a code a simple method that allows ignoring fine points of seismic design, compensates therefore by slightly raising the base shear coefficient or lowering the computed capacity, and sets strict limits of applicability. One such method was introduced in 1966 but its use began a few years earlier, when the code was essentially complete. Experience with the method has been satisfactory although we must admit that there have been no truly severe earthquakes that would put it through a decisive test. California engineers have objected to what they consider oversimplifications in the method on the basis of local experience. A more convincing calibration must await further experience in the Federal District.

Static Method

Base Shear Coefficients.--If one does not wish to compute the fundamental period of vibration the base shear coefficient for computation of deformations is C, that is, 1/g times the spectral acceleration in the flat portion of the spectrum for the site in question. However, one must compute deflections anyway except when using the simplified method of analysis. Hence it is ordinarily not much more work to compute the fundamental period of vibration using Schwartz' quotient (13),

$$T = 2\pi\left(\frac{\sum M_i y_i^2}{\sum F_i y_i}\right)^{1/2} \tag{3}$$

where F_i, M_i, and y_i = mass, applied force, and computed deflection of ith floor, respectively. If it is found that $T_1 \leq T \leq T_2$, the base shear is not modified, if $T \leq T_1$, it is taken proportional to the corresponding spectral ordinate, and if $T > T_2$, it is reduced as described under the next heading.

In shear structures the base shear coefficient never exceeds 1/g times the spectral acceleration ordinate associated with the fundamental period provided the acceleration spectrum for shorter periods does not fall above the hyperbola passing through the point mentioned (13). The criterion for $T \leq T_2$ is thus conservative for shear structures. For flexural structures it is approximately correct if the spectrum is flat for periods smaller than the fundamental; not so if spectral acclerations are a decreasing function of period for periods smaller than T. Hence the need for a different provision when $T > T_2$.

The base shear coefficient for computing lateral forces is equal to the one for computing deformations divided by the ductility factor.

Distribution of Accelerations for Computation of Story Shears.--The shear distribution in practically all buildings and most other structures is reasonably approximated by specifying horizontal accelerations proportional to elevation above ground, to the square of this elevation, or intermediate between these. No sufficiently simple yet satisfactory criterion has been devised to decide on this shape in every given instance. In general, the longer the fundamental period of vibration the more important will flexural deformations tend to be relative to shear deformations and the more significant will the contributions of higher modes tend to be relative to the fundamental. If spectral accelerations decrease with period, the base shear coefficient in flexural structures usually exceeds 1/g times the fundamental-period spectral accelerations. These trends are reflected in the provision that, if $T > T_2$, horizontal accelerations for deflection computations be assumed equal to the sum of a term proportional to elevation and a term proportional to the square of elevation and such that the corresponding base shear coefficients equal $(T_2/T)^k\{1 - k[1 - (T_2/T)^k]\}C$ and $1.5k(T_2/T)^k[1 - (T_2/T)^k]C$ respectively, where $-k$ = exponent of T in expression for spectral accelerations when $T \geq T_2$. Thus the acceleration distribution passes smoothly from a straight line when $T = T_2$ to a parabola as T tends to infinity, while the total base shear coefficient varies smoothly from C when $T = T_2$ toward $1 + k/2$ times the spectral ordinate at T tends to infinity. (See Ref. 13 for results of analysis of chimney stacks.)

Story Torques.--The design torsional eccentricity is to be taken as the most unfavorable of $e_s - 0.1b$ and $1.5e_s + 0.1b$, where e_s = statically computed eccentricity and b = story dimension measured perpendicularly to direction being analyzed. Coefficient 1.5 intends to take into account dynamic magnification (13). The additional $\pm 0.1b$, known as "accidental eccentricity," is introduced to cover eccentricities due to discrepancies between the mass, stiffness, and resistance distributions used in analysis and true distributions at the time of a strong earthquake; torsional oscillations induced by a rotational component of ground motions; and other sources of torsion not considered explicitly in analysis. This is admittedly a crude way of dealing with so many variables, but for one thing a more ambitious provision would meet objections and probably rejection by practicing engineers, and for another, the present state of knowledge does not justify a more refined treatment. For example, it is known that torques induced by the rotational ground-motion component in a one-story symmetrical building are a decreasing function of the building's fundamental period of vibration (13). Accidental eccentricities to reflect this part of the phenomenon have been suggested, ranging from $\pm 0.05b$ for flexible buildings to $\pm 0.1b$ for the more rigid ones (13). In multistory, uniform shear-structures this type of accidental torque is roughly independent of elevation above ground (8), so the corresponding accidental eccentricity, expressed in terms of b, should increase with elevation. Effects of discrepancies between actual and computed rigidities depend on structural layout and on structural materials; they are likely to be considerably greater in reinforced concrete structures and in those having masonry shear walls than in steel moment-resisting frames, and they are likely to be a decreasing function of the number of columns in the story under consideration. The dynamic amplification of 1.5 is open to discussion since, on the one hand, when $e_s << b$, the amplification factor can considerably exceed 1.5 but, on the other hand, when this factor is large the story shears are smaller than computed under the assumption that story torques are nil (13).

The 1957 regulations specified that the design story torque not be taken smaller than half of the maximum in stories above nor the design eccentricity smaller than half the maximum in stories below. This was intended to reflect torsional oscillations in one story due to eccentricities elsewhere in a structure. For instance, eccentricities in lower stories that support a symmetrical tower will cause the tower to oscillate in torsion; yet this will go undetected in a conventional static analysis. Eccentricities in upper stories may cancel each other in such an analysis, leaving lower stories apparently free of torsion; yet the upper-story eccentricities will generally induce torsional oscillations in the entire structure. This provision was adopted years later in the New Zealand Code. In the 1966 Mexican provisions it was discarded because it seemed an unwarranted complication. An attempt was made to reintroduce it in 1976 but met with objections because it requires additional accountancy and having raised the accidental eccentricity from $\pm 0.05b$ to $\pm 0.1b$ was felt by many to cover most cases in which eccentricity in remote stories might have a significant effect. The provision was accordingly left out of the new code.

Overturning Moment.--Main reasons for using design overturning moments smaller than the integral of the story-shear envelope are (13), a) maximum story shears do not occur simultaneously at all stories; b) the envelope of story shears used in design is approximate; if correct at one elevation and not unsafe at others, it is practically sure to be conservative throughout most of the structure's height; c) nonlinear behavior of soil near the soil-structure interface and separation over a small portion of this surface decreases overturning moments at and near the structure's base; and d) it is often considerably more expensive to resist overturning moments at the structure's base than it is to resist the associated story shears, so that, from the viewpoint of optimum design, overturning moments at the base should be reduced. Reductions are not, however, so pronounced as many codes of the past allowed. The 1976 document allows 20% reduction at the base, none at the top, and a linear interpolation between bottom and top reduction factors. Additionally, $M \geq hV$, where M = overturning moment, h = distance to center of gravity of portion of building above elevation being considered, and V = story shear at that elevation. A much more drastic reduction is not justified. The provision that $M \geq hV$ insures that equilibrium will be satisfied (there should be no reduction in the uppermost story from conditions a, c, and d) and protects structures having unfavorable mass distribution along their height.

It might be appropriate to allow a slight further reduction as one moves from immediately above to immediately below the foundation. It was felt, though, that the phenomena involved were not sufficiently understood to permit drafting a general provision on this matter and that allowing a favorable contact-stress distribution to be assumed between structure and soil took care of at least the major part of the phenomena in question.

Local Accelerations.--Local accelerations specified in the code equal $(A = a)g$, where Ag = local acceleration for story shear computations and ag = design acceleration for infinitely rigid structures. Horizontal accelerations assumed for computation of story shears are not the maximum local accelerations, with the possible exception of the uppermost floor (roof). As we approach a structure's base, accelerations for computing story shears go to zero while local accelerations should tend to the maximum ground acceleration. In addition, appendages and the like dynamically magnify the local accelerations. Provisions covering the gamut of conditions of practical interest in a theoretically sound, accurate manner would be very extensive, as a great many variables are involved. On the other hand, neither moderate overdesign nor moderately increased failure probabilities are seriously objectionable since initial cost and consequences of failure are orders of magnitude smaller than for the entire structure. Hence the simple provision in the code. This provision recognizes the tendency for local accelerations to increase with elevation above ground because of structural oscillations.

Vertical Accelerations.--Horizontal ground motion can induce nonneglible vertical accelerations in portions of a structure. Such is the case with the top of inverted pendulums, parts of the roof system in some mill buildings, and the upper floors of tall buildings exhibiting significant overall flexural deformation. In all these cases local vertical accelerations combine to produce rotational inertia. The code provision is a simple device that recognizes dynamic magnification much in the way in which this is done for story torques. It is required that, where computed vertical displacements exceed some limit, vertical accelerations be taken into account equal to 1.5 times the design horizontal acceleration times the ratio of local vertical to horizontal displacements.

Through this provision and that of P-δ effects there is an increase in design forces in recognition of vulnerability of inverted pendulums. These are now to be designed in a more realistic way than was specified in previous codes, which merely called for a uniform increase in base shear coefficient for structures of this type.

Modal Analysis

Degrees of Freedom per Story.--In buildings having sufficiently rigid floor diaphragms the structure may be modeled with one degree of freedom per story independently of the magnitude of statically computed torsions. It may seem strange that the code should not specify explicit dynamic consideration of story torques when torsional eccentricities are large relative to base dimensions. The fact is that dynamic magnification of torsional eccentricities is a decreasing function of e_s/b (13), so such a provision would not be justified.

Story Torques.--It is permitted to take story torques into account using the criterion specified for static analysis.

Combination of Modal Responses.--If S_i denotes the computed response in the ith natural mode of vibration, the design response is to be computed as

$$S = \sqrt{\sum S_i^2} \tag{4}$$

unless the frequencies of two or more natural modes that contribute significantly in Eq. 4 are close together. When this is the case the code itself does not specify a formula. It refers to a supplementary document issued by the Federal District Department, which contains expression

$$S = \left(\sum_i \sum_j \frac{S_i S_j}{1 + \varepsilon_{ij}^2} \right)^{1/2} \tag{5}$$

in which

$$\varepsilon_{ij} = \frac{\omega_i' - \omega_j'}{\zeta_i' \omega_i + \zeta_j' \omega_j} \tag{6}$$

where ω_i = undamped circular frequency of ith natural model, $\omega_i' = \omega_i \sqrt{1 - \zeta_i'^2}$ = damped circular frequency of ith natural mode, $\zeta_i' + 2/\omega_i s$, ζ_i = damping ratio of ith natural mode (assumed equal to 0.05 unless a different value is justified), and s = duration of segment of stationary white noise equivalent to the family of actual design earthquakes (s is to be assumed as 20, 30, and 40 sec in zones I, II, and III, respectively. For zone IV when not reclassified on the basis of local information of soil properties, s = 50 sec). Equations 5 and 6 are justified in Ref. 13. Equation 5 simplifies into Eq. 4 when all the ω_i's are sufficiently different from each other so that $\varepsilon_{ij} \gg 0$ when $\omega_i \neq \omega_j$.

Reasons for presenting Eqs. 5 and 6 in a separate document are that they look complicated and it is not unlikely that better practical procedures will be developed in the near future.

Use of Eqs. 5 and 6 is necessary in structures whose coupled torsional and translational natural modes have some frequencies close to each other or in which appendages bring about this condition.

Quantities that are ordinarily computed through Eqs. 4 and 5 are not those used directly in design. We compute by means of these expressions such responses as story shears and torques and overturning moments; in design we use maximum generalized member forces: shears, bending moments, and axial forces in beams and columns. There is not a one-to-one relation between both types of response, so in principle we ought to use Eqs. 4 and 5 to compute design responses proper. However, savings in computation time are significant and loss of accuracy is normally not, and errors are always on the safe side, so that only in special cases is the more precise approach justified.

Step-by-Step Analysis

Analysis of buildings subjected to specified ground motions has rarely if ever been used for design in any country. Yet it seems desirable to leave this possibility open. The first objection to the method lies in that dispersion of responses is large; hence, several independent ground motions should be processed, which entails high computer costs. (The 1976 code calls for at least four representative, independent motions.) Second, it is not a trivial matter to verify that the motions are consistent with other code requirements. Third, as a consequence of nonlinear behavior the effects of gravity and of various components of ground motion must be analyzed simultaneously. Fourth, as the 1976 code requires, one should take into account uncertainty in structural parameters. This is the easiest objection to overcome if we introduce some simplifying assumptions, such as complete correlation among structural stiffnesses and validity of a second-moment approximation (18). The first assumption allows recognizing randomness of stiffnesses by merely changing the time scale in the acceleration records whose effects we analyze. The second assumption implies that it suffices to analyze the structure for the records having the original time scale multiplied by 1 + V and by 1 - V, where V = coefficient of variation of natural periods. We immediately obtain the expected values of structural responses and their coefficients of variation. An approximate method (14) then leads to the optimum design.

Concluding Remarks

Participation in the drafting of building codes has been a powerful stimulus to research and innovation. The informality with which cultural and economic factors have made Mexican engineers regard building codes, more as guidelines than as rigid norms, has had its negative aspects but the freedom it has bred has promoted experimentation and rapid evolution.

A critical examination of present codes discloses their weaknesses and thereby points out fruitful areas of research. Interaction between code writing and research has thus proved to be a fecund network of intellectually rewarding activities.

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Table 1. Acceleration Spectrum Parameters

Zone	a	C	T_1	T_2	k
I	0.030	0.16	0.3	0.8	1/2
II	0.045	0.20	0.5	2.0	2/3
III	0.060	0.24	0.8	3.3	1

a = spectrum ordinate at T = 0 for ordinary buildings, divided by g

C = ordinate of flat portion of spectra for ordinary buildings, divided by g

T_1 = T at beginning of flat portion, in sec

T_2 = T at end of flat portion, in sec

k = exponent of T_2/T in descending branch

Table 2. Allowable Ductility Factors (Abridged)

1. Allowable Ductility Factor: 6.0. Buildings having moment resisting steel or concrete frames. Well defined yield point. Compact steel sections. Helically reinforced concrete columns or equivalent. Load factor 1.4 for brittle modes of failure. Special design at plastic hinges. $\nu_{min}/\bar{\nu} \geq 0.80$.
2. Allowable Ductility Factor: 4.0. Buildings having moment resisting or braced steel, timber, or concrete frames or concrete shear walls provided unaided frames can carry at least 25 percent of horizontal forces. $\nu_{min}/\bar{\nu} \geq 0.65$.
3. Allowable Ductility Factor: 2.0. Buildingshaving timber, concrete, or confined solid masonry resisting elements.
4. Allowable Ductility Factor: 1.5. Buildings having the foregoing structural systems and reinforced or confined hollow masonry shear walls that comply with certain limitations.
5. Allowable Ductility Factor: 1.0. Other buildings; other structures; other structural materials.

ν = safety factor for story shear; ν_{min} = minimum in entire building; $\bar{\nu}$ = average for all stories.

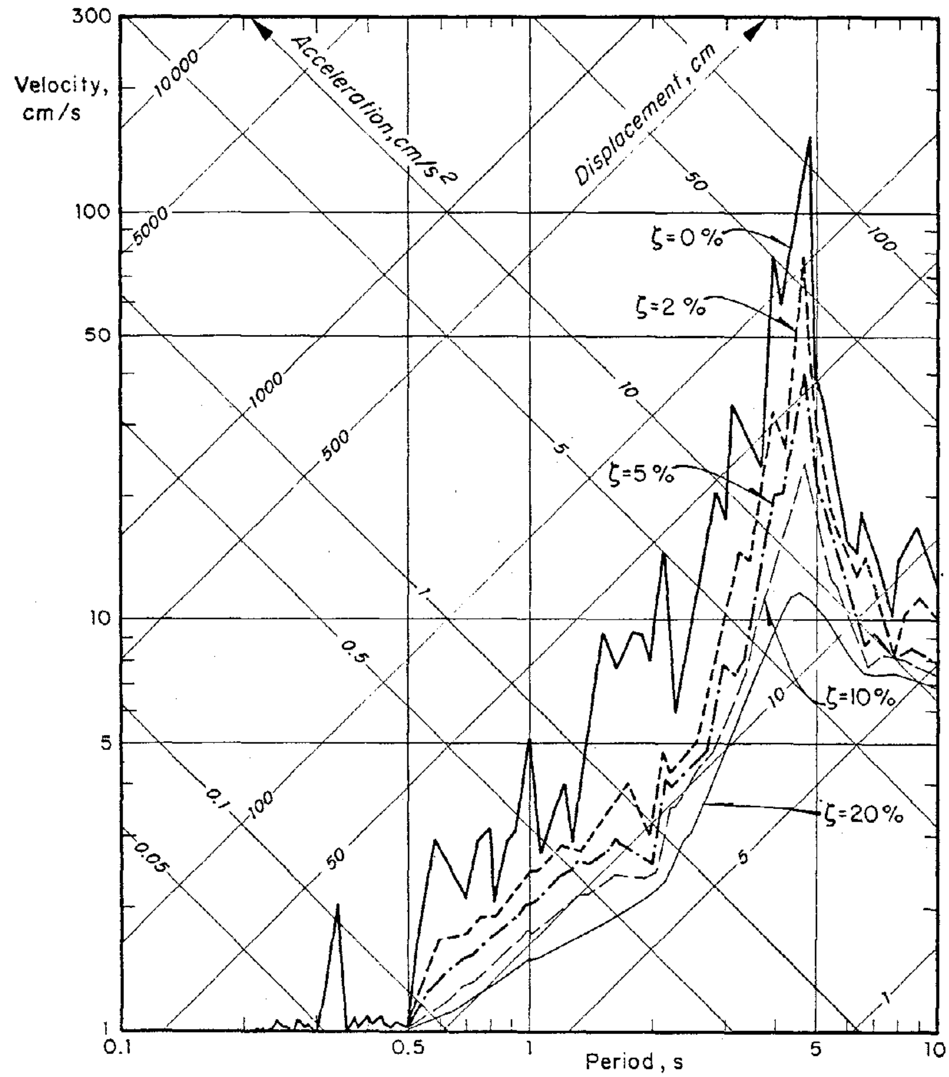

Fig 1. Acceleration spectrum at Sports Palace (10)

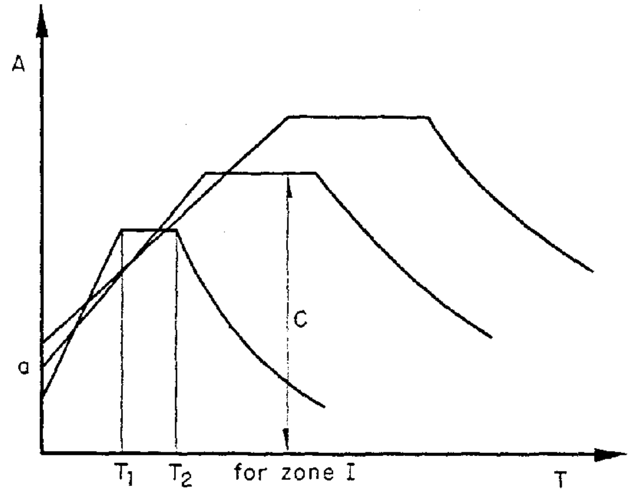

Fig 2. Design spectra for computation of deformation

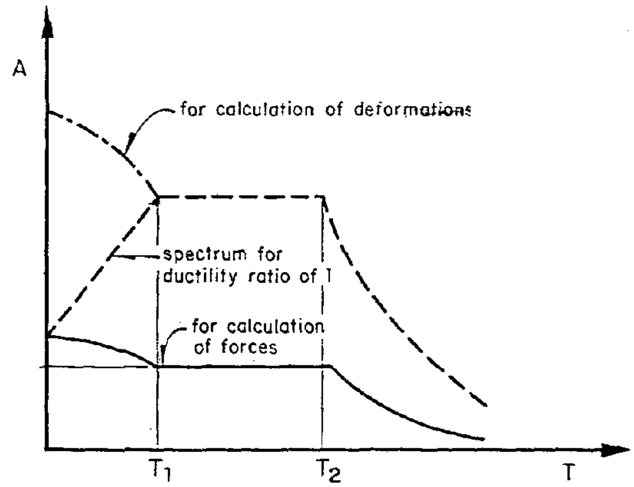

Fig 3. Effects of ductility on design spectra

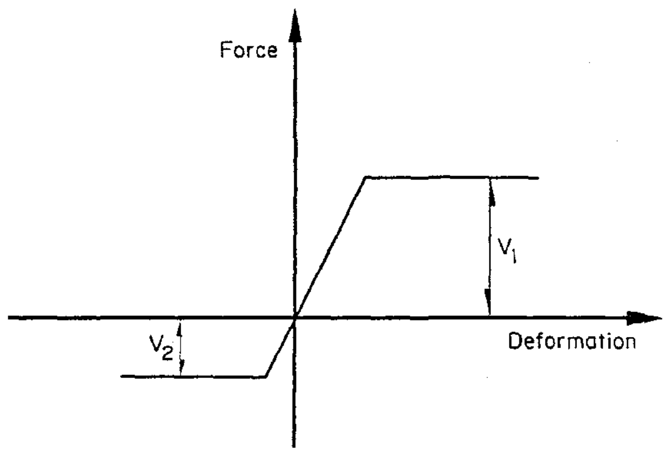

Fig 4. Asymmetric force – deformation curve

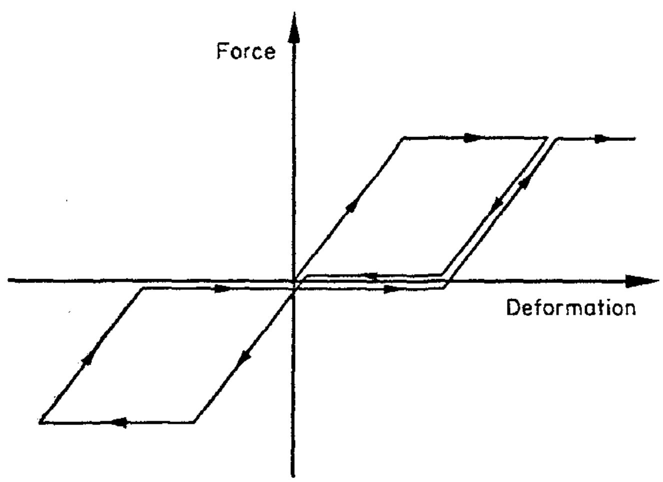

Fig 5. Force – deformation curve for ductile braced structures

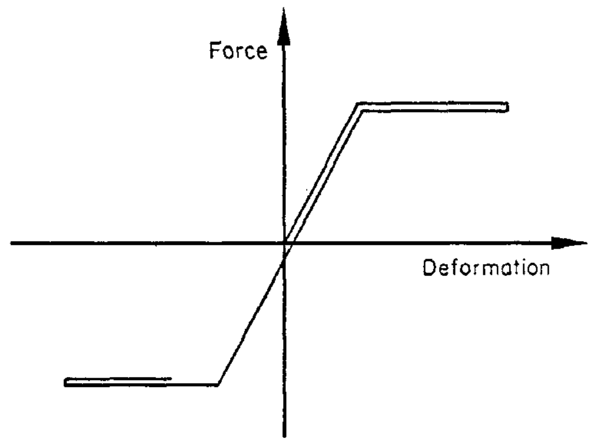

Fig 6. Elastic nonlinear force–deformation curve

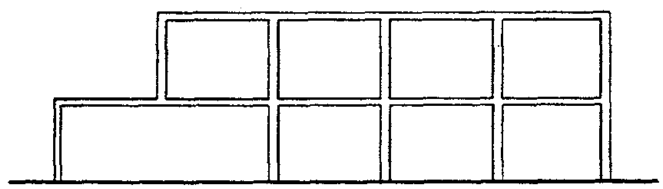

Fig 7. Structure exhibiting asymmetrical force–deformation curve

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED
CONCRETE BUILDING CONSTRUCTION (ERCBC)
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EARTHQUAKE RESISTANT REINFORCED CONCRETE BUILDINGS IN MEXICO: RESEARCH NEEDS AND PRACTICAL PROBLEMS

Luis Esteva*

INTRODUCTION

This paper is complementary of Ref. 38. Together, both papers intend to describe the state of knowledge and practice of earthquake resistant design of reinforced concrete building construction in Mexico. Ref. 38 deals with the technical bases for the recently promulgated regulations (1), while the present paper concentrates on research needs and design practice achievements and weaknesses.

Development of seismic design technology has been largely based on engineering judgement and interpretation of observed response of structures during severe shocks. Seismic design coefficients have reached their present values through successive approximations to what engineers and code writers have implicitly deemed optimal, and many limitations of design norms in force at a time have been disclosed during strong earthquakes. As the characteristics of new constructions depart from the former ones, direct extrapolation of past experience is of more limited value and support of basic knowledge is required. The practice of seismic design of reinforced concrete building construction is affected by technical progress in a variety of fields, of which the most significant can be broadly grouped into behavior of reinforced concrete members and systems, dynamic response, structural analysis, optimum design decisions and innovative design. Some of these fields cover problems specific to reinforced concrete structures, while others are significant for other type of construction as well.

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BEHAVIOR OF REINFORCED CONCRETE MEMBERS AND SYSTEMS

Modern design codes, in particular those in force in Mexico at present, explicitly deal with ductility concepts. Design spectra are rationally derived from linear response spectra for given values of ductility factors; but these factors have not stemmed from detailed knowledge of the probability distributions of ductilities that can be developed by different kinds of structures. Instead, they were indirectly obtained after establishing design coefficients and spectra that seem acceptable in the light of the seismic performance of a number of structures designed in accordance with given prescriptions. The fact that actual structural performance is usually obscured by the participation of non-structural elements and the need to extrapolate ductility-based design requirements to new classes of structural systems make it desirable to acquire a detailed understanding of behavior of members, systems and local details when subjected to high stress reversals.

Ribbed flat plate systems

Owing to its light weight, ribbed flat plate construction is extensively used in buildings resting on the soft clay formation of Mexico City. The floor system consists of an orthogonal grid of small reinforced concrete ribs (depths range from 30 to 50 cm, and widths from 8 to 15 cm; center to center distance between ribs ranges from 70 to 150 cm) and a continuous flange of the same material connecting their upper portions. In the immediate vicinity of the intersection of the columns with the floor system the assembly of ribs and flange is replaced with a rectangular portion having the same depth as the ribs (see Fig. 1). Design and analysis specifications are those developed for solid flat plates; their validity should be questioned, as differences between relative values of bending and torsional stiffnesses of both types of systems may cause marked discrepancies between values of panel stiffnesses to be used in the lateral load analysis of structural frames and in the evaluation of displacements. These differences are surely reflected also in the internal stresses within the panels.

The most critical concept in the earthquake resistant design of these systems is their capacity to provide continuity between columns and panel elements for the types of internal forces produced by lateral loads. As in ordinary flat plates, failure may take place by diagonal tension along a critical section that lies around the intersection of the column and the plate; but in this case an additional mode of the same type must be considered, now with the critical section lying along the perimeter of the constant thickness portion adjacent to the column.

Tests on solid flat plates show that shear failure arround columns tends to dominate over bending moment, on account of the latter being distributed over a large width (34). This makes difficult to obtain large ductilitity values, even when failure is initiated by yielding at those sections where bending stresses are highest. The problem persists in riboed flat plates. Ductility factors adopted in Mexican regulations for design of the systems do not differ from those used in reinforced concrete beam-column frames; however, adequacy of those factors and of the strong-column-weak-girder concept should be verified in the light of tests of the systems under consideration under the action of severe alternating lateral loads.

Ductility of reinforced concrete shear walls

The capacity of slender reinforced concrete shear walls has been attested by laboratory experiments (2). The possibility of buckling of the wall free edges along the story height is the main hindrance to the mentioned capacity; the problem is usually circumvented by providing the wall edges with stiffening elements running along them. Because of the significant architectural advantages of using unstiffened walls, the influence of slenderness on their response to cyclic applications of bending moments and shear-forces in the wall plane should be studied. Additonal questions should be posed concerning the significance of the interaction of these forces with the lateral sway of the wall produced by first-order displacements normal to the wall.

Low-aspect-ratio shear walls

Most experimental research on the cyclic-load performance of reinforced concrete shear walls has been caried out on walls with high ratios of total height to width (aspect ratios). These tests have shown that overall bending and diagonal tension capacities can be evaluated by conventional ultimate load criteria usually applied to reinforced concrete beams. Overall bending under the action of forces in the wall plane can then be forced to be the critical failure mode through adoption of adequate safety factors with respect to that mode and to other potential modes such as edge buckling or diagonal tension; but for aspect ratios smaller than about 1.5, conventional beam design is no longer applicable and the probability of failure in shear is overwhelmlingly larger than in bending, unless shear load factors are increased much above ordinary levels. Available results show that failure patterns in the walls under consideration deviate significantly from those taking place in high-aspect ratio walls; reinforcement patterns required to attain some degree of ductility should reflect qualitative differences in behavior.

High-ductility reinforced concrete structures

Mexican regulations require that seismic design spectra be obtained by adequate reductions of specified elastic response spectra, and the reductions imply ductility factors ranging from 1 to 6. Ordinary reinforced concrete frame buildings are assigned a ductility factor of 4. This figure can be raised to 6 provided some special requirements are fulfilled: that all columns have transverse reinforcement capable of confining the core to a degree comparable to that of a standard helix; that load factors for shear and diagonal tension, torsion, instability and other forms of brittle failure be taken as 1.4, instead of 1.1 specified for the combination of permanent and accidental loads; that provisions be adopted to permit the formation of the number of plastic hinges necessary for the development of ductile story behavior in a plastic collapse mechanism should lateral forces be high enough, and that safety factors with respect to story shears be essentially uniform throughout the structure. At plastic hinges, longitudinal reinforcement is required to have a defined yield zone; yield stresses higher than 4200 kg/cm^2 (60,000 psi) are not permitted. Some of these regulations are probably too stringent, in particular the limit to yield stress of reinforcement at plastic hinges, the value of 1.4 proposed for the load factor associated with some failure modes and the need to use helically reinforced columns throughout the structure. Comparisons of preliminary designs for ductility factors of 4 and 6 do not show any substantial saving in the initial cost for structures designed and detailed in accordance with the requirements specified for the higher value; hence there is little motivation to go through the additional requirements. Nevertheless, because well detailed structures are bound to suffer little damage, economic advantages associated with lower expected costs of failure and damage may constitute the most significant asset of the high ductility alternative. The extent of this advantage has not been evaluated, nor the possible reduction in ductility because of release in restrictions.

Interaction between reinforced concrete frames and infill diaphragms

Reinforced concrete frames infilled with clay-brick or hollow-concrete-block diaphragms provide an efficient manner of resisting seismic-forces. Although at the cost of extensive damage, these diaphragms have shown their capacity to absorb energy during strong earthquakes; but this capacity rests on the development of interaction stress between diaphragm and frame (Fig. 2). When a system of lateral loads acts on an infilled frame, tensile stresses develop at some regions in the diaphragm and at some portions along the frame-diaphragm interface. Tensile capacity is reached at low lateral forces, and cracking first occurs there. The diaphragm then acts essentially as a compression strut, with its ends thrusting upon the frame corners, creating a complex pattern of internal stresses. Further increase in lateral loads may give place to diagonal cracking of the diaphragm, and then, eventually, of the frame corner. In the latter case, the system's capacity suddenly drops. However, if cracking of the frame conrner is prevented, the system can still undergo significant additional deformations at essentially constant load. Deterioration of the energy absorbing capacity for further loading cycles strongly depends on the behavior of the frame corner. Hence the convenience of formulating adequate design and detailing criteria.

System identification

Present design coefficients and safety levels are largely the result of engineering judgement and interpretation of the response of structures to severe ground shaking. Rational criteria for predicting structural responses and for establishing optimum values of the design parameters have undergone a drastic evolution in the last few years, and the computation capabilities have not lagged behing; but our limited knowledge about the properties of structural systems precludes taking full advantage of those criteria and capabilities. Response spectra for different damping ratios and ductility levels for given accelerograms can be obtained at the touch of a button, and with comparable ease can the response history of a nonlinear system with arbitrary damping matrix be determined; but the properties of actual systems, that should be fed into the magnificent computer programs available, are far from known. Such apparent contradiction finds its justification in the difficulties inherent to determination of the relevant properties of structural systems for a wide range of stress levels, including those produced by destructive earthquakes.

Extrapolation of past experience about seismic response of structures does not suffice for predicting behavior of those to be built, as constructive practices, structural concepts and architectural details evolve rapidly. Research should cover determination of natural frequencies and damping ratios for more than one natural mode, shaking-table tests on large models, deployment of strong motion-instrument arrays on important structures and analysis of the records obtained during severe ground shaking. Instrumentation of models and actual structures should be such as to permit determination of force-deflection curves for a number of structural assemblages at different stress levels; quantitative criteria suitable for practical applications should be formulated aiming at the definition of these curves for first load application and of the laws governing their degradation under subsequent load cycles.

However significant the recording of the response of full-scale structures can be, analysis of its outcome is not an easy undertaking: to mention just two factors contributing to the difficulty, nonlinear behavior is influenced by interaction between structural responses to several simultaneous components of ground motion, and separation of the contributions of "structural" and "architectural" elements to the mechanical properties of a system is practically impossible. Therefore, the role of the studies on the seismic response of full-scale structures should be complementary to that of model testing: the latter should produce criteria for defining the mechanical properties under discussion and the former should aim at calibration of these criteria and at correlating structural response with consequences, including human distress and economic loss.

Non-standard details

Stress paths and stress concentrations at joints between reinforced concrete structural members are complex and difficult to estimate at the design stage, and so are the laws governing concrete behavior under such patterns. In the absence of factual evidence or simple criteria, design of joints and special details is seldom based on more than intuition. Limitations of such a practice remain often concealed for years until an earthquake discloses them. In some instances negative consequences stem from sheer negligence, while in others they can be ascribed to ignorance. Not even such standard details as the interior beam column connection had been studied until a few years ago (3) and it was no small surprise discovering all the precautions necessary for the design and construction of sufficiently strong and ductile unconfined knee-joints (4); much more difficult are problems arising from the behavior of less efficient and more complicated types of connections (Fig. 3) whose use will not be abolished, however troublesome it is to structural engineers, as the exterior configuration of these connections is often essential to the architectural project. Given that those details will remain in use, we ought to try to understand their behavior and produce design recommendations. Studies should include cyclic load tests at high stress levels and finite elements analysis of stress distribution for first load application.

Small scale models

Detailed knowledge about materials behavior and capacity to produce analytical models of complex systems does not suffice to eliminate the need for performing tests on physical models. Because costs of laboratory tests grow very rapidly with specimen size, small scale models are mandatory. Important conclusions concerning the static behavior of shear wall systems have been obtained by tests on models to scales in the order of 1:4 (35). For smaller scales, representativity of models is not perfectly understood, although some comparisons between the responses of model and prototype under monotonous loading are encouraging (36). The possibility of extending this optimism to alternating loads must await additional studies.

STRUCTURAL ANALYSIS

Despite existence of extremely prowerful computer programs for structural analysis, designers often face lack of practical tools applicable to some special structures, as well as adequate criteria for defining mechanical properties of members or degrees of fixity in their foundation. Some of these problems are particular to the structural systems specific to reinforced concrete, while others arise with other materials as well.

Foundation of shear walls and cores

Stiffness of shear walls and cores strongly depends on that of their foundations; the usual hypothesis that those elements are embedded on a perfectly rigid base is acceptable when that base is a very stiff formation or a group of piers or piles, but may furnish unacceptably high system stiffnessess and unrealistic distributions of lateral forces among frames and walls for some buildings resting on the soft clay formation in the Valley of Mexico. Buildings not taller than six to eight stories are usually founded by partial compensation of their weight and those having up to about 15 stories are founded on friction piles. A "rigid" box constituted by an orthogonal gridwork of girders and upper and bottom slabs provides control of differential settlements. Bases of shear walls and cores are embedded on the girders; support flexibility depends on the local foundation-box deformability and on its interaction with the soil. Numerical tools required for the theoretical study of this problem are available (5, 6); there is need only to explore the significance of the problem for various ranges of the relevant parameters and to produce practical rules applicable in design practice.

Seismic settlements of structures founded on soft soil

Limitation of differential settlements constitutes a basic criterion for the design of foundations on compressible soil. Although the matter is seldom explicitly dealt with in quantitative terms, and although procedures of structural analysis that account for the interaction between foundation and soil are less frequently applied for the estimation of differential settlements under permanent loads, the latter are intuitively controlled by adopting foundation depths and cross sections deemed to have shown satisfactory performance under similar conditions; but differential settlements produced by seismic forces and their influence on stress distribution throughout the system are usually not evaluated.

An even more difficult problem of analysis arises when overturning moments are high enough to lead to a transient situation where compressive stresses acting at the soil-foundation interface are overcome within part of the interface and the foundation acts as a cantilever subjected to uplifting forces transmitted by the columns (Fig. 4).

In-plane deformability of floor diaphrams

Buildings having narrow or highly irregular plans and irregularly distributed stiffnesses of lateral-force-resisting elements can pose special problems of structural analysis: the in-plane deformability of floor diaphragms can be of the same order as that of the vertical elements, and the usual hypothesis that the diaphragms are infinitely rigid no longer holds. As a consequence, shears acting on stiffer elements are lower than those predicted under the hypothesis mentionded, and the reverse is true for the more flexible ones. Rough estimates of the deviations in some particular instances attest to their practical significance; however, no clear rules have been furnished to designers concerning the ranges of variables within which the effect can be discarded, and application of available computer programs that idealize each diaphragm as an assemblage of finite elements would entail inadmissible computer costs. An alternative method of analysis has been envisaged, based on defining the in-plane deformed configuration of each diaphragm by a small number of parameters; but the detailed formulation and implementation of the method are yet to materialize.

Irregular building plans

Flat plate construction offers significant advantages for moderately tall buildings in Mexico; especially important among them is the flexibility that it permits in the location of columns, which do not need to be aligned according to two orthogonal sets of frames (Fig. 5). But this apparent advantage has turned into drawback, as abuse of flexibility has led to extremely irregular plans where concepts such as center strip and column strip, ordinarily used as basis for defining the distribution of bending moments, cease to exist. In the lack of simple methods of analysis the mentioned distribution is ordinarily estimated on the basis of very crude assumptions and at the expense of undersirable cracking. Here again, availability of finite element computer programs capable of handling these problems has not satisfied the designer's need of a compromise solution between crude assumptions and computer costs.

Analysis of ordinary beam-column systems where frames are contained in oblique planes has received no better attention than the foregoing problem.

DUCTILITY DEMAND

Use of ductile materials is no token for good seismic performance.

Parallel to studies oriented to determining local and overall capacities of structural details, members and systems to dissipate energy through ductile hysteretic behavior, significant efforts must be devoted to understanding the parameters that affect the distribution of ductility demands throughout a structure and to develop prescriptions to account for those demands in design practice.

Extensive research has been conducted in the last few years, aiming at estimating ductility demands of nonlinear structures. In nearly all cases, systems studied have been intended to represent building frames, either as assemblages of beams and columns or as shear systems (7-12) individual members or sections are assumed to behave in accordance with bilinear hysteretic models. Results show that as a rule local ductility demands are not uniform throughout the building height, that their distribution is influenced by those of stiffnesses and strengths, and that sensitivity thereto is higher for shear systems than for beam-column assemblages. Other load deflection curves have been studied (13-16) and their results used to assess the range of validity of ductility based lateral force reduction criteria and to formulate special requirements applicable to various types of load-deflection curves. For instance, the latest Mexico City seismic design regulations (1) make ductility factors— and hence seismic design coefficients— depend on the variability of actual safety factors throughout the building height. This variability is usually ascribed to some stories being stronger than specified, as a consequence of architectural requirements or presence of non-structural elements. For a ductility factor of 6 to be applicable, the requirement is imposed that the minimum safety factor with respect to shear at any given story not depart by more than 20 percent from the average of the corresponding safety factors in the other stories; a similar provision exists for the ductility factor to be taken equal to 4, but with the 20 percent restriction released to 35. The influence of a large number of significant variables is neglected, because of lack of systematic studies.

Structural framing and ductility demands

There is no reason why the distribution of ductility demands found valid for building frames that respond essentially as shear systems should be applicable to structures characterized by different types of stress paths. An obvious example of the discrepancies to be expected is found in the ductility demands at the ends of beams impinging on the edges of a slender shear wall: owing to overall bending deformations of the wall, beam end rotations and vertical deflections are negligible near the wall bottom and increase substantially as one moves towards the top. Accordingly, bending moments predicted by linear structural analysis at the ends of the mentioned beams are negligible near the bottom and inadmissibly high near the top. Not infrequently designers explicitly recognize the need for permitting the concentration of ductile deformations at some spots, instead of trying to design for the internal stresses predicted by linear analysis; this practice entails substantial differences in ductility demands throughout the building height, over and above those to be expected as a consequence of the inadequacy of linear methods of dynamic analysis in the case at hand. In addition, there is practically no information about ductility demands on the wall itself. The situation is no better when one talks about a number of usual structural systems, namely braced or trussed systems, chimney stacks and many others where overall bending deformations are significant.

Force-deflection curves

Differences in global or local force-deflection curves usually stem from differences in materials used, but can also result from differences in type of framing. Design requirements of modern codes are largely based on results of dynamic-response analysis of elasto-plastic systems. Not much attention has been paid to studying the implications of other types of behavior on optimum design coefficients. Other pertinent load-deflection idealizations are depicted in Fig. 6. Lateral strengths required for not exceeding given ductility demands in these systems are as a rule greater in 10 to 50 percent than those valid for the conventional elasto-plastic system (13-16).

In the asymmetric elasto-plastic case, yield strength is different in each direction of load application. This happens, instance,because gravity loads increase or decrease the lateral capacity of the second story of the system shown in Fig. 7, depending on whether the vertical reaction to force Q_2, transmitted to beam AB at O, is directed upwards or downwards. Ref. 13 shows that ductility demands can be greater by about 50 percent than those obtained for a conventional elasto-plastic system having a lateral capacity equal to that of the weakest branch of the asymmetric one. These increments have been explicitly accounted for at least in one design code (17), by means of the factor $(1 + 10r)/(5 + 6r)$ (r equals the ratio between larger and smaller lateral capacities of a given story in the direction of analysis) to be applied to the lateral force coefficient.

Slip-type curves (Fig. 6) usually arise when lateral loads are carried by elements such as slender cross braces or tie cables, which can only carry tensile forces. Associated increments in ductility demands estimated with the criterion of equal strain energy (18) have led to corrective factors for the lateral force coefficient of up to 1.7. Further studies (19) have shown that global ductility demands can be kept essentially equal to those of the conventional reference system if 25 percent of the total lateral capacity is provided by a system that responds elasto-plastically.

Yielding elastic curves constitute close approximations to the behavior of some prestressed concrete beams subjected to antisymmetric end moments; these curves have very narrow hysteretic loops (15). Mexican regulations call the attention to the problem, but do not provide expressions to cover the resulting enhancement of response.

Degrading curves (Fig. 6), are frequently found in systems where a significant portion of the lateral capacity resides in brittle materials and where no special precautions have been taken to prevent excessive damage in each cycle of load application. Such is the case, for instance, in poorly detailed reinforced concrete frames. Differences in the influence of stiffness degradation for reinforced concrete structures design and built under different specifications are reflected in Mexico City's 1976 regulations: ductility factors of 6 or 4 are specified depending on a number or conditions, one of which is the adoption or not of a set of reinforcement details inspired on those proposed by ACI as mandatory on structures to be built in seismic areas; another is that the higher ductility factors cannot be adopted for structures with shear walls, diaphragms or bracing members. When there are elements of these latter types, use of ductility factors of 4 requires that the strength of the frames themselves be at least 25 percent of the total.

Slenderness effects

Unstable curves (Fig. 6) can result from the action of large vertical loads on deformed structures. Influence of instability effects on ductility demands and on safety against collapse can be more significant than that associated with the features of the curves previously discussed, and is usually controlled in design practice by the specification of amplification factors for lateral deflections and internal forces that account for increments associated with second order effects. Mexican regulations propose that story sways corresponding to design lateral forces and multiplied by the ductility factor be amplified by the factor $(W/h)/(R/Q - 1.2\ W/h)$, obtained in Ref. 20, as an approximation valid for frame systems subjected to static lateral loads; h is story height, R story stiffness, Q ductility factor (hence, R/Q = equivalent story stiffness), and W is weight of the portion of building above the story considered. Dynamic responses of a number of multistory elasto-plastic shear systems have been computed using a step-by-step procedure for the two alternate assumptions of considering and neglecting slenderness effects, and for three accelerograms recorded on soft soil in Mexico City. Ratios of story sways for these two assumptions show that amplification factors can be in some cases much larger than computed by the expression given above, particularly for structures having natural periods shorter than those corresponding to the peaks of acceleration spectra (21). Such conclusions were obtained even for lateral stiffnesses significantly greater than the limits impli ed by the restrictions imposed on story drifts. The phenomenon is probably, though, of little consequence as its effects seem to be covered by the ordinates of the design spectrum, characterized by a flat constant-ordinate region for a wide range of periods shorter than that corresponding to the spectral peak. It is believed that the problem has not been sufficiently studied.

The question of residual displacements has to be looked at in the framework of slenderness effects.

Ductility and base rocking

Soil-structure interaction can be represented approximately by a foundation mass supported on a set of linear springs and dampers. Overall system deflections are the result of structural deformations and foundation displacements. Under strong earthquakes, structural members develop as a rule a more pronounced degree of non linearity than the foundation, and to a nominal value of overall system ductility demand there corresponds a higher value for the structure. Soil-structure interaction increases a system's natural periods and damping, but often the net result is the occurrence of large ductility demands than if the soil were perfectly rigid. A preliminary study (22) shows that under extreme conditions ductility demands can increase several fold, particularly at the top stories of tall buildings, which suggestis the preponderance of foundation rocking. The problem may have been overemphasized, however, because tall buildings will usually be founded on piles, and rocking loses importance.

A preliminary draft of the present code called for a reduction to allowable ductility values in terms of the ratio between contributions of structural and foundation deformations to the top deflection. Its promulgation was deferred for need of response analyses and cost-benefit studies.

Ductility and torsion

Observations about the seismic performance of some asymmetric structures suggest that excessive amplification of torsional oscillations may result when nonlinear structural behavior gives place to eccentricities that grow with response level (23). The influence of this property on ductility demands has been studied using step-by-step analysis of several multistory asymmetric shear buildings (24). Lateral stiffness and strength were provided by pairs of shear diaphragms located at the building ends and oriented in the direction parallel to ground motion. Buildings were symmetrical as regards mass distribution, but not with respect to stiffnesses. The diaphragms were designed according to the results of modal analysis for the spectral ordinates corresponding to a number of accelerograms and given ductility levels. Load factors were taken as unity. Then the stiffer diaphragm was redesigned for a higher load factor, aiming at producing a structure with the property mentioned above, and the dynamic responses for the original and the modified systems were computed. Contrary to expectations, no increase in the ductility demand in the softer diaphragm resulted from overdesigning the stiffer one; this may be due to increased capacity for direct shear.

Response to two-dimensional excitation

Interaction between stresses and strains produced by two orthogonal components of ground motion gives place to interactinn between the available ductilities of frames parallel to these components. When ductility develops essentially in the girders, as expected when design follows the strong-column-weak-girder concept (10), interaction takes place only at the bottom of ground-floor columns; neither its influence on the laws governing the distribution of ductility demands in complex systems nor its significance in design have been assessed.

Response of nonlinear structures on soft soil

Practically all research concerning the computed seismic response of nonlinear systems has used accelerograms obtained on firm soil or simulated records inferred from them. Relations between the responses of linear and nonlinear systems are doubtless sensitive to variables such as shock duration and frequency content. Adequate relations should be established for a wide range of ground conditions.

SYSTEMS FOR SHOCK ISOLATION AND ENERGY ABSORPTION

Research and practice in earthquake engineering have been mainly oriented to the detailed study of conventional solutions. They have concentrated on devising procedures for the prediction of structural response and on providing the cross sections and reinforcement patterns required for strength and ductility; innovation has been nearly non-existent.

The idea of isolating structures from the ground motion can be traced back to the flexible first story concept (25), which never became practical, on account of the problems posed by slenderness effects. The idea has been revived in the last few years (26, 27), and a variety of systems have been proposed capable of deforming laterally without taking significant loads and without the risk of instability failure; such are, for instance, roller bearings (28), hanging supports (29) and pads built with very flexible materiales (30). Absolute isolation is not desirable because relative displacement between building and foundation would be excessive and because of the need to resist wind loads. Use of additional elements has been suggested, acting in parallel with the motion isolators and capable of providing hysteretic damping and small lateral strengths (31). Numerical studies of structural response have been conducted (32, 37) and preliminary designs of particular systems obtained aiming at studying costs and benefits (32). Initial cost must be compared with the savings from reductions in strength requirements for the structures and with the long term advantages of improved seismic performance.

Only one case is known to the author where a system of the type just described has reached the construction stage: it is a three-story school building whose column footings rest on beds of steel balls. Inmediate research programs foresee placement of strong motion instruments in this building and its foundation, as well as shaking-table tests on models of various types of shock isolators.

Use of removable devices intended to dissipate energy at the expense of damage has been advocated, but seldom implemented, probably for lack of data concerning their behavior.

DESIGN OPTIMIZATION

Engineering design is rooted on society's need to optimize. It implies considering alternate lines of action, assessing their consequences and making the best choice. In earthquake engineering, every alternate line of action includes adoption of both a structural system and a seismic design criterion, while assessing its consequences requires estimating structural response and hence the expected cost of damage. The choice is based on a comparison of initial, maintenance and repair costs for the various alternatives. This implies the prediction of probability distributions of costs of damage in structural and nonstructural members for given intensities.

Structural safety

Nominal values of design variables, safety factors and implicit safety levels have been traditionally established by trial and error and engineering judgement. The theory of structural reliability has provided the framework for recent attempts to attain consistency among those rules and to extrapolate them to more general conditions. Simplified formulations derived from the basic concepts have led to design criteria which approach consistency while not departing from the simplicity required by practical applications. Nominal values of design variables are chosen so that the probability that each variable will adopt a more unfavorable value does not exceed a certain limit, which is related to the overall probability of failure for the complete system. Relations between member and system safety depend on the types of potential failure modes and on the probability distributions of the mechanical and geometrical properties involved, including spatial correlations among them. While a reasonable stock of information is available concerning the distributions of individual variables, little is known about their correlation. This lack of information can be ascribed to the fact that it has never been asked for, as little effort has been spent on research on system safety.

Design criteria for the revision of safety conditions are usually stated in terms of the ratio of structural capacity to internal load at each individual critical section. The effect under study must hence be accounted for by making required safety factors vary with the number of critical sections involved in a failure mode. This is the basis for the prescription in the 1976 Mexico City code (1) stating that the generalized force acting on every shear wall or column that takes up more than 20 percent of the story generalized force (shear, torque or overturning moment) be increased 20 percent; or by the prescription concerning non redundant systems in ATC recommendations (33) stating that when a building system is designed or constructed so that the failure of a single member, connection or component would endanger the stability of the building, that member, connection or component should be provided with a strength at least 50 percent greater than required otherwise. Discrepancies between the two criteria described are sufficiently important to justify a detailed study of the problem. An obvious weakness of both is that they ignore the influence of member size (i.e. a column vs a shear wall) in the uncertainty associated with its strength.

Cost-benefit studies

Relations between structural response, damage and repair cost find applications in determining expected costs of damage for given intensities. The latter information is of use in formulating seismic design decisions and in estimating expected costs per unit time, for insurance purposes. Formal application of cost-benefit studies to decision making in earthquake engineering is often hindered by problems that arise in evaluating material losses given structural response or in the expression of different types of failure consequences in the same unit or, more specifically, in assigning monetary values to concepts such as panic, loss or prestige, or injury and death. In spite of these difficulties, qualitative conclusions can be derived from informal comparisons of initial investments and expected performance. Progress in this area is desirable, as the influence of these criteria on initial and deferred costs can be quite significant. Future research should cover the definition of failure consequences and calibration of the results of theoretical cost-benefit models with those of intuitive, informal optimization.

CONCLUDING REMARKS

Modern reinforced concrete buildings must be designed for earthquakes according to criteria that account for nonlinear behavior and ductility demands. Available ductility is very sensitive to reinforcement details and to ratios of safety factors with respect to different failure modes. Needed research encompasses a variety of fields, such as determination of stress — strain laws for structural assemblages under alternating cycles of severe loading, study of nonlinear response of systems with different stress — strain laws, influence of soil-structure interaction, torsional-oscillations and slenderness effects on the mentioned response, development of systems for shock isolation and energy absorption, and basic studies on structural safety and design optimization.

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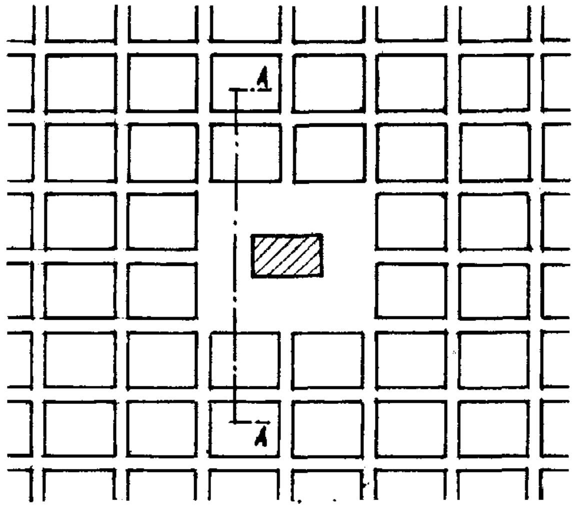
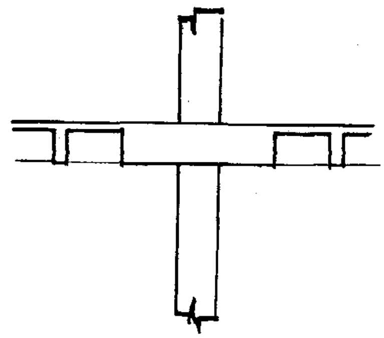


a) Plan View



b) Section A-A

FIG. 1 RIBBED FLAT PLATE

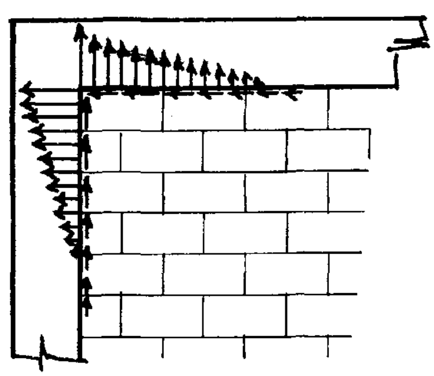


FIG. 2 INTERACTION STRESSES

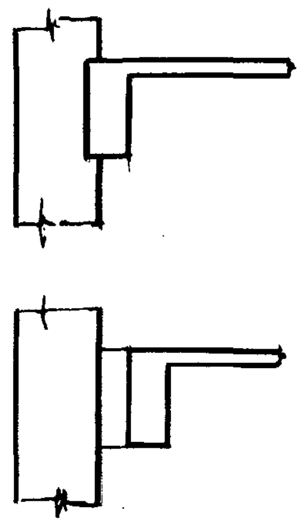


FIG. 3 ECCENTRIC CONNECTIONS

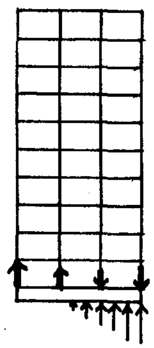


FIG. 4 FOUNDATION UPLIFTING

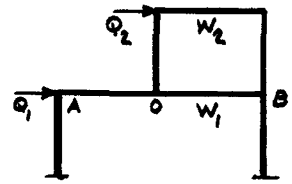


FIG. 7 SPECIAL STRUCTURE

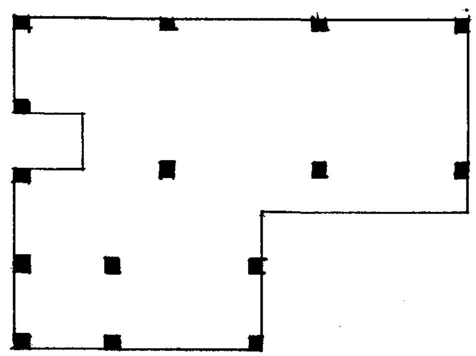
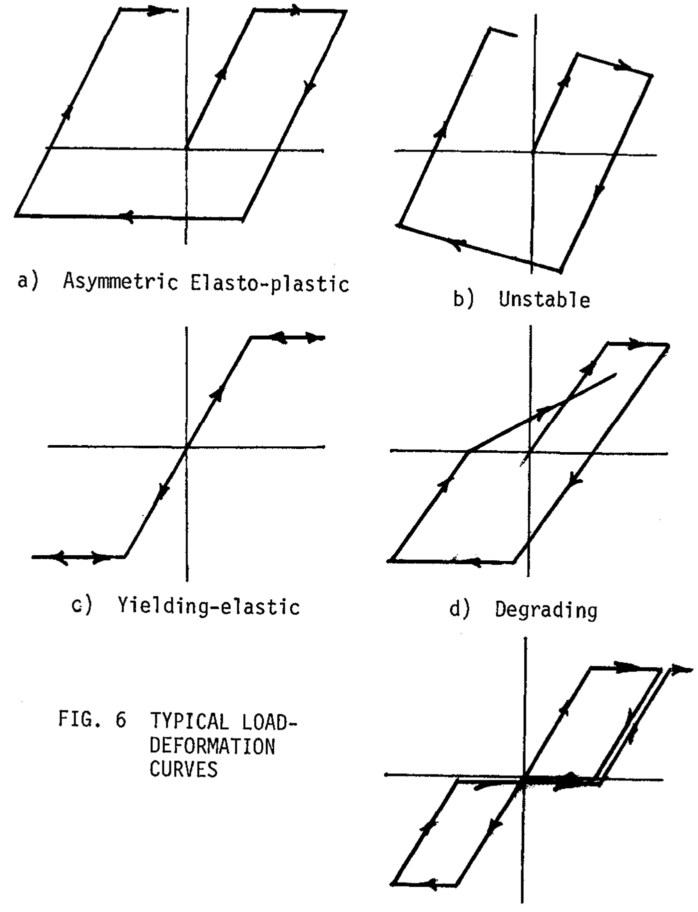


FIG. 5 IRREGULAR BUILDING PLAN



a) Asymmetric Elasto-plastic

b) Unstable

c) Yielding-elastic

d) Degrading

e) Slip-type

FIG. 6 TYPICAL LOAD-DEFORMATION CURVES

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

ACCOMPLISHMENTS AND RESEARCH AND DEVELOPMENT NEEDS IN NEW ZEALAND

by

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INTRODUCTION

New Zealand is situated in part of the seismically active circum-Pacific belt. The early settlers, coming from non-earthquake countries, introduced few measures for earthquake resistance in their buildings. However, the Napier earthquake of 1931 caused the collapse of many masonry buildings and was responsible for the loss of some 256 lives. This event resulted in a shift in emphasis of building type from load bearing masonry to framed buildings. Codes for earthquake resistant design in New Zealand have gradually evolved since the Napier earthquake. During the last decade, particularly, much attention has been given to earthquake engineering, and seismic provisions now dominate most design procedures in New Zealand. The Standards Association of New Zealand has the responsibility for the issue of design codes.

The New Zealand Society for Earthquake Engineering was formed in 1968. Since 1974 it has been the New Zealand National Society for Earthquake Engineering. The aim of the Society is the advancement of the science and practice of earthquake engineering. Engineers, scientists, architects, contractors, and all who have an interest in earthquakes and their effects, are eligible for membership. The Society publishes a quarterly Bulletin containing a wide range of papers on earthquake engineering, and has organised conferences on earthquake engineering which have been of great value to the profession in New Zealand and neighbouring countries.

A good deal of research has been conducted in New Zealand into aspects of earthquake resistant design. Most of the experimental research has been conducted at the laboratories of the University of Canterbury, University of Auckland, the Ministry of Works and Development, and the Department of Scientific and Industrial Research. The analytical research has been conducted by those organizations along with major consulting engineering firms. Most of this research has involved reinforced concrete structures, because reinforced concrete is the most used building material in New Zealand mainly for reasons of economy and availability of local materials.

This paper reviews the current situation regarding codes for the seismic design of concrete structures in New Zealand, and discusses aspects of research and development which have been conducted in this country during recent years. Areas of difficulty where further research is required are emphasized.

CODES FOR THE DESIGN OF EARTHQUAKE RESISTANT CONCRETE BUILDINGS IN NEW ZEALAND

Code for General Structural Design and Design Loadings for Buildings

The current New Zealand code covering general requirements for structural design and design loadings for buildings, NZS 4203 [1] was published in 1976. This Code allows design by the "strength method" or by the "working stress method". The working stress method is referred to as the "alternative method" in the Code to emphasize that the strength method is to be preferred.

For the strength method the load factors have been derived from ACI 318-71 [2]. The design loads U involving combinations of service dead load D, reduced service live load L_R, and earthquake load E, should not be less than which ever of the following combinations gives the greatest effect:

$$U = 1.4D + 1.7L_R \tag{1}$$

$$U = 1.0D + 1.3L_R + E \tag{2}$$

$$U = 0.9D + E \tag{3}$$

The reduced service live load L_R is found by multiplying the service live load by a reduction factor which depends on the use of the building and the tributary area of floor or roof supported by the structural member. In equivalent static force analysis the total horizontal seismic force V on the building is given by:

$$V = C_d W_t \tag{4}$$

where W_t is the total reduced gravity load equal to service dead load plus a proportion of the service live load (typically one-third of the service live load), and

$$C_d = CISMR \tag{5}$$

where C is the basic seismic coefficient which varies between 0.15 and 0.05, depending on the seismic zone (there are three seismic zones in New Zealand), the period of the structure, and the subsoil condition; I is an importance factor which varies between 1.6 and 1.0, depending on how essential it is that the building should be functional after a seismic disaster; S is a structural type factor equal to 0.8 for ductile frames with an adequate number of possible beam plastic hinges and ductile coupled shear walls, and having a higher value for less ductile structural types; M is a material factor equal to 1.0 for reinforced concrete and 1.2 for prestressed concrete; and R is a risk factor equal to 1.0, unless the building accommodates large numbers of people or contains high risk chemicals or other materials when a greater value is used. For buildings with equal floor loads the distribution of V up the height of the structure is triangular with the greatest horizontal load at the top, except that for buildings with a height to width ratio greater than 3, $0.1V$ is considered concentrated at the top storey and the remaining $0.9V$ is distributed up the height. To provide for shear resulting from torsional motions the applied horizontal force at each level is applied eccentrically with respect to the centre of rigidity at that level. Two equations for the eccentricity of the horizontal load are given, each a function of the horizontal dimension of the building and the distance from the centre of rigidity to the centre of mass, and the most unfavourable condition is used.

By way of general seismic design principles the Code requires that buildings should be designed to be capable of dissipating significant amounts of energy inelastically under earthquake attack. Buildings designed for ductile flexural yielding should be the subject of "capacity design" and should have "adequate ductility". In the "capacity design" of earthquake resistant structures "energy dissipating elements or mechanisms are chosen and suitably designed and detailed, and all other structural elements are then provided with sufficient reserve strength capacity to ensure that the chosen energy dissipating mechanisms are maintained throughout the deformations that may occur." An approximate criterion for "adequate ductility" given in the commentary of the Code is that "the building as a whole should be capable of deflecting laterally in at least eight load reversals so that the total horizontal deflection at the top of the main portion of the building under the loading of Eqs. 2 and 3, calculated on the assumption of appropriate plastic hinges, is at least four times that at first yield without the horizontal load carrying capacity of the building being reduced by more than 20%. The horizontal deflection at the top of the building at first yield should be taken as that when yield first occurs in any main structural elements or that at the earthquake load E calculated on the assumption of elastic behaviour, whichever is the greater."

The Code recognizes that it is reasonable to design the beams of two-way frame systems for seismic loading considered separately along each of the two principal axes of the structure. However it requires that columns or walls, including their foundations and joints, which are part of a two-way system should be designed for the concurrent effects resulting from a general direction of seismic loading which causes the simultaneous yielding of all beams framing into the column or wall in the two directions.

Ductile frames, according to the Code, should be capable of dissipating seismic energy in a flexural mode at a significant number of plastic hinges in the beams, except that energy dissipation by column plastic hinge mechanisms is permitted in single or two-storey structures and in the top storey of multistorey structures. Apart from those specific cases, columns should be designed to have adequate overcapacity to avoid column hinge mechanisms, taking into account possible distributions of column moments which may be different from that derived from elastic analysis, and column loads appropriate to the simultaneous formation of plastic hinges in beams in several storeys.

Ductile coupled shear walls, according to the Code, should be designed so that the coupling beams yield before the walls, and the coupling beams should be detailed so as to be capable of dissipating significant seismic energy. Only when the yield capacity of the coupling beams, associated with the major portion of the overturning moment on the structure, is exhausted should the wall elements yield. Ductile cantilever shear walls should be designed to ensure that energy dissipation is by flexural yielding and that the wall will not fail prematurely in a non-ductile manner.

Code for Reinforced Concrete Design

The current New Zealand code for reinforced concrete design, NZS 3101P [3], was published in 1970. This Code is based mainly on ACI 318-63 [4]. For ductility provisions for seismic design reference is made in the Code to the 1968 SEAOC recommendations [5] with some additional requirements concerning the anchorage of beam bars in external columns, ignoring the shear force carried by the concrete in potential plastic hinge zones, and a number of other factors. This New Zealand Code is now significantly out of date and it has been the practice of designers in New Zealand in recent years to use ACI 318-71 [2] with its Appendix A for seismic provisions or a more recent issue of the SEAOC recommendations.

At the present time the New Zealand concrete design code is being revised. The new code will be based mainly on the 1977 version of ACI 318, with additional seismic provisions based on recent research and experience in New Zealand and elsewhere. The revised form of the New Zealand concrete design code should be ready for circulation for comment in New Zealand in late 1977. The background to these seismic design provisions will be discussed later in this paper.

Code for Prestressed Concrete Design

The current New Zealand code for prestressed concrete design, NZSR 32 [6], was published in 1968. This Code is also based mainly on ACI 318-63 [4] and contains no recommendations for seismic design. The Code is being revised at present. The prestressed concrete provisions will be placed in the same code as those for reinforced concrete and will also be ready for circulation for comment in New Zealand in late 1977.

SOME COMMENTS ON GENERAL RESEARCH AND DEVELOPMENT IN NEW ZEALAND INTO THE SEISMIC RESISTANCE OF STRUCTURAL CONCRETE

Much research into the seismic resistance of structural concrete has been conducted in New Zealand in recent years, mainly in the laboratories of the University of Canterbury, University of Auckland, the Ministry of Works and Development, and the Department of Scientific and Industrial Research. The experimental work has involved properties of steel and concrete under seismic type loading; shaking table tests on model structures; pseudo-static load tests on reinforced concrete model frames and shear walls, beam-column subassemblages, slab-column subassemblages and slab-wall subassemblages; and pseudo-static load tests on prestressed concrete beam-column subassemblages. The experimental work has been accompanied by analytical studies involving dynamic response of various structural systems to severe seismic ground motions, and theoretical studies of moment-curvature behaviour and strength and deformation characteristics of the range of structures considered. Most of this work has been reported in the literature and that dealing with reinforced concrete is summarized in a recent book [7]. The outcome of the recent work will be discussed in the following sections of this paper.

An activity which has been of great value during 1976 and 1977 has been a series of meetings organized by the New Zealand National Society for Earthquake Engineering. The Society is intending to hold a series of Seismic Design Workshops for structural designers in New Zealand to make them more familiar with the new Loadings Code [1] and with recent developments in seismic design procedures for reinforced concrete. In the first instance the Workshops will concentrate on reinforced concrete framed structures. It was realized however that there are many "grey areas" where there are almost as many views as there are designers ; for example, capacity design procedures, beam-column joint design, design of plastic hinge regions, protection of columns, anchorage of bars, etc. Therefore the first step taken was to organize a series of meetings of a Preworkshop Discussion Group made up of a number of structural engineers from consulting organizations, the Ministry of Works and Development and the Universities to attempt to obtain a consensus view on the range of issues. This Group has now had four meetings of one or two day duration and has achieved a remarkable degree of agreement on most issues. Background papers in the form of design recommendations and commentary were written by individuals and these papers were discussed and modified at the meetings until agreement was reached. The topic areas of these papers are: general analysis of frames [8], torsion of frames [9], design for beam flexure [10], evaluation of column actions [11], design for column flexure and axial load [12], design for shear on beams and columns [13], design of beam-column joint cores [14], and parts and secondary elements [15]. It is expected that the papers in these areas will be published in the June and September 1977 issues of the Bulletin of the New Zealand National Society for Earthquake Engineering. The papers will be under the authorship of individuals but they represent the consensus view of the Group. Two further topics, foundations and low ductility frames, will be the subject of further meetings of the Group. The results of this series of meetings of the Preworkshop Discussion Group has been invaluable to the Concrete Design Committee of the Standards Association of New Zealand which has been preparing the new concrete design code. Some of the design recommendations arising from the Group discussions are outlined later in this paper.

DESIGN CRITERIA FOR DUCTILITY DEMAND

General

The designer who thinks fundamentally will have difficulty in deciding the level of ductility necessary at critical sections of the members of earthquake resistant structures designed to code loading. Codes have been vague on this point and definitions of "ductility factor" have been various and confusing. Nonlinear dynamic analyses of code-designed structures responding to typical severe earthquake motions have given an indication of the order of postelastic deformations required,but the number of variables is so great that no more than qualitative statements can be made at present. Computer programs capable of nonlinear dynamic analyses of reinforced concrete framed structures have been developed in New Zealand, for example [16,17,18,19]. Computer programs for the nonlinear dynamic analysis of structures involving shear walls have recently been developed, for example Taylor [20]. Valuable studies have been conducted using these programs. For prestressed concrete only single degree of freedom systems have been analysed in New Zealand [21,22,23], but such studies using hysteresis loop shapes which model the load-deflection behaviour of prestressed and reinforced concrete systems have shown that a prestressed concrete system when responding to a severe earthquake will be subjected to a maximum displacement of about 30% more than that of a reinforced concrete system with the same strength, initial period of vibration and viscous damping ratio. Such studies have resulted in the Loadings Code [1] allocating a material factor M = 1.2 to prestressed concrete, compared with M = 1.0 for reinforced concrete, when determining the seismic coefficient. The Loadings Code [1] indicates that a displacement ductility factor in the order of 4 is required of ductile earthquake resistant structures and the detailed provisions for section design, in order to achieve this displacement ductility factor, are left to the Concrete Design Code.

Confusion has existed in the minds of some designers regarding the definition of ductility factor, since it can be expressed in terms of displacements, rotations or curvatures. The displacement ductility factor $\mu = \Delta_u/\Delta_y$, where Δ_u = maximum lateral deflection and Δ_y = lateral deflection at first yield, is the value commonly determined in nonlinear dynamic analyses. Some dynamic analyses have determined the rotational ductility factor of members θ_u/θ_y, where θ_u = maximum rotation of end of member and θ_y = rotation at end of member at first yield. The information needed by the designer concerns the required member section behaviour expressed by the curvature ductility factor ϕ_u/ϕ_y, where ϕ_u = maximum curvature at the section and ϕ_y = curvature at the section at first yield. Thus the required ϕ_u/ϕ_y value is a far more meaningful index for ductility demand than the other possibilities. It needs to be recognised that there can be a significant difference between the magnitudes of the displacement, rotational and curvature ductility factors. This is because once yielding has commenced in a structure the deformations concentrate at the plastic hinge positions and further displacement occurs mainly by rotation of the plastic hinges. Thus the required ϕ_u/ϕ_y ratio will be greater than the Δ_u/Δ_y ratio [7].

When calculating ductility factors the definition of first yield deformation (displacement, rotation or curvature) often causes difficulty when the load or moment-deformation curve is not elastoplastic. This may occur for example due to plastic hinges not forming simultaneously in members, or to longitudinal bars in reinforced concrete members at different depths in the section yielding at different load levels. In such a case it is suggested that the "first yield" displacement be taken as the displacement calculated for the structure assuming elastic behaviour up to the strength of the structure in the first load application to yield, as illustrated in Fig. 1. A similar definition can be adopted for first yield rotation and curvature. Sucl
Such a definition for first yield allows comparison of the effects of different loop shapes with the same initial stiffness and strength on the ductility demand.

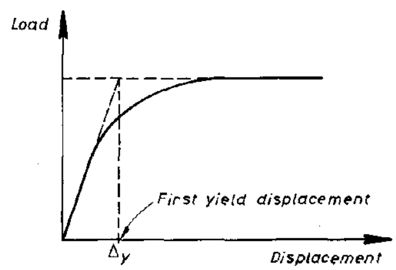


Fig. 1 Possible Definition for "First Yield" Displacement When Load-Displacement Relationship is Curved.

It is evident that the sequence of plastic hinge development in structures will influence the curvature ductility demand. Nonlinear dynamic analyses have indicated that ductility demand concentrates in the weak parts of structures and that the curvature ductility demand there may be several times greater than for well proportioned structures. This can also be illustrated by examination of static collapse mechanisms. Fig. 2 shows a frame and shear walls which can be used for seismic resistance. Possible

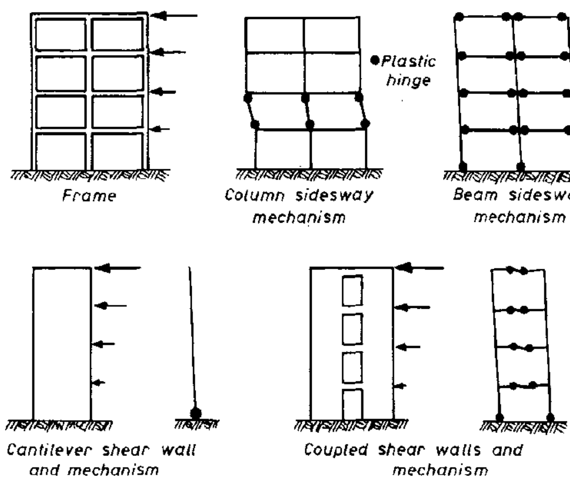


Fig. 2 Building Structures Under Seismic Loading and Possible Mechanisms.

mechanisms which could form due to flexural yielding and formation of plastic hinges are also shown in the figure. If yielding commences in the columns of a frame before the beams,a column sidesway mechanism can form. In the worst case the plastic hinges may form in the columns of only one storey, since the columns of the other storeys are stronger. Such a mechanism can make very large curvature ductility demands on the plastic hinges of the critical storey [7], particularly for tall buildings. On the other hand if yielding commences in the beams before in the columns a beam sidesway mechanism, as illustrated in Fig. 2, will develop [7], which makes more moderate demands on the curvature ductility required at the plastic hinges in the beams and at the column bases. Therefore a beam sidesway mechanism is the preferred mode of inelastic deformation, particularly since the straightening and repair of columns is difficult. Hence for frames a strong column - weak beam approach is advocated to ensure beam hinging. In the actual dynamic situation higher modes of vibration influence the moment pattern and it has been found [18] that plastic hinges in the beams move up the frame in waves involving a few storeys at a time. For cantilever shear walls the static collapse mechanism involves a plastic hinge at the base and the curvature ductility demand for a given displacement ductility factor depends very much on the plastic hinge length as a proportion of the wall height. For coupled shear walls the mechanism shown in Fig. 2 can occur [7] and ideally the beams should yield before the wall bases to enable easier repair. The static collapse mechanisms of Fig. 2 are idealized in that they involve behaviour under code type static loading. The actual dynamic situation is different, due mainly to the effects of higher modes of vibration, but nevertheless considerations such as in Fig. 2 give the designer a reasonable feel for the situation.

Required Agreement and Further Research

It is apparent that agreement needs to be reached on the various definitions of ductility factor to avoid the looseness of definition which exists at present. It is also evident that many more nonlinear dynamic analyses need to be conducted on a range of building types using a variety of strong motion records to ascertain the likely curvature ductility demand on the critical sections to allow the designer to anticipate ductility requirements with more confidence.

LOADING CRITERIA IN PSEUDO-STATIC LOAD TESTS

General

A great deal of valuable information on the effects of severe earthquakes has been obtained from tests on structural subassemblages in the laboratory using cycles of pseudo-static loading. Structural subassemblages rather than complete structures have normally been tested due to difficulties with size. Fig. 3 shows a test specimen representing a beam-column joint of a frame. The

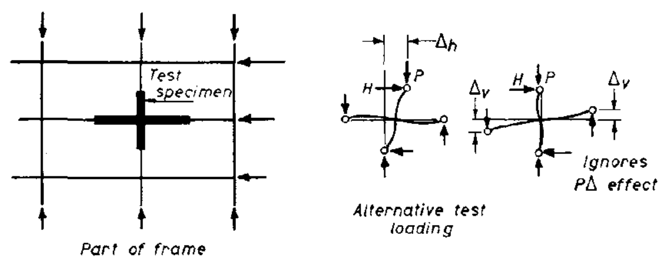


Fig. 3 Beam-Column Joint Test Specimen and Load Application

members extend approximately to the points of contraflexure. It is impossible in such tests to simulate accurately all aspects of loading and ductility demand of the actual more complex structure. However if the loading sequence is severe enough, and if the strength, stiffness and energy dissipation of the test specimen are found to be acceptable, satisfactory performance of the actual structure can be confidently expected. In the past a variety of loading sequences and acceptance criteria have been used by various research laboratories throughout the world, making the comparison of results difficult and resulting in different conclusions from tests being reached.

Two loading criteria which have been used in New Zealand laboratories in pseudo-static load tests are shown in Figs. 4 and 5. The displacement

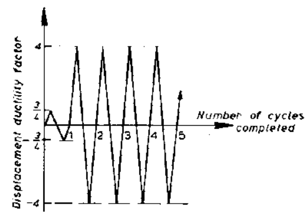


Fig. 4 Simple Loading Criterion

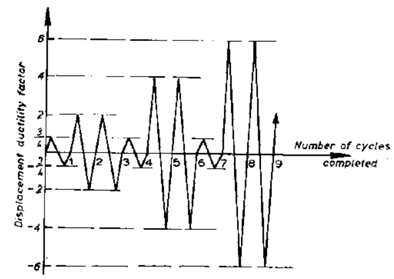


Fig. 5 More Complex Loading Criterion

ductility factor is calculated using the first yield displacement for the first inelastic load run defined as in Fig. 1. The simple loading criterion shown in Fig. 4 involves initial loading runs in the elastic range to establish the initial elastic stiffness and then four loading runs in each direction to displacement ductility factors of 4. This criterion follows that recommended in the New Zealand Loadings Code [1]. The more complex loading criterion shown in Fig. 5 involves more elastic loading runs to observe stiffness changes between the cycles of imposed inelastic deformations, and cycles of imposed inelastic deformation with gradually increasing displacement ductility factor. It is suggested that the loading criterion shown in Fig. 5 be adopted since it allows observation of behaviour at various levels of imposed ductility during the test. A simple acceptance criterion is that the seismic load carrying capacity should not reduce by more than 20% during the test [1].

Required Agreement and Further Research

The chosen magnitude of the imposed displacement ductility factor, the number of cycles of loading, and the centre of oscillation of the deflections, are debatable issues which can only be answered in detail by those who have conducted extensive nonlinear dynamic analyses. However there is no doubt that agreement needs to be reached on a standard loading criterion for pseudo-static load tests so that test results can be compared on a consistent basis. Also, agreement needs to be reached on an acceptance criterion for structural behaviour.

RESEARCH INTO MATERIAL BEHAVIOUR

Steel and Concrete

The response of structural concrete to seismic loading depends very greatly on the stress-strain characteristics of steel and concrete. Cyclic load tests have been conducted in New Zealand on typical samples of reinforcing bar and prestressing wire [24,25,23,21,26] and idealizations proposed to model the stress-strain curves. The idealizations have used mainly the Ramberg-Osgood equation with empirical values for the constants in the equation .

Available experimental data for the monotonic stress-strain characteristics of concrete confined by rectangular hoops or by circular spirals has been examined and idealizations proposed for the stress-strain curves of concrete so confined [24,27,40]. The idealized monotonic curve has been assumed to act as the envelope curve for cyclic loading. Idealizations for the hysteresis loops within the envelope curve have also been proposed for cyclic loading [24,28,26]. However the experimental evidence is limited mainly to concrete specimens containing simple square hoops or circular spirals. There has been very little testing of concrete confined by the more complex arrangements of transverse reinforcement typical of columns in practice.

Such material research is fundamental to work on strength, ductility and seismic response. The strength and ductility of a structure depends on the actual stress-strain characteristics of the materials. The seismic response of well detailed structures depends on the moment-curvature characteristics of the critical sections, which can be determined from the material properties [28,26,23].

Required Further Research

More testing is required to clearly define the full stress-strain relationships for the various grades of steel in use in New Zealand and overseas so as to give statistical information on actual yield strengths, strain hardening characteristics, and cyclic loading behaviour. Also, more experimental work is required on confined concrete specimens, particularly with various arrangements of overlapping rectangular hoops, rectangular hoops with supplementary cross ties, and circular spirals, to establish with better accuracy the empirical parameters which define the monotonic and cyclic stress-strain characteristics.

BEAMS IN FRAMES

General Aim

Research into the behaviour of beams under seismic type loading in New Zealand, and recommendations for the design of beams, have aimed at producing detailing procedures which will result in ductile flexural behaviour at potential plastic hinge regions and will prevent other non-ductile types of failure which lead to strength and stiffness degradation.

Longitudinal Steel Content in Reinforced Concrete Beams

The seismic provisions of ACI 318-71 [2] require that the tension steel ratio should not exceed 0.5 of that producing balanced flexural failure, and that at column faces the positive moment capacity of beams should be at least 0.5 of the negative moment capacity. It can be shown that this will ensure an available curvature ductility factor ϕ_u/ϕ_y of at least 6 for an extreme fibre maximum concrete strain of 0.004 [7]. Hence if the curvature ductility factor demand is 2 or 3 times this value, as is likely in a severe earthquake, the concrete needs to be confined effectively by closely spaced stirrup ties to prevent crushing of the core concrete at high strains, and damage to the cover concrete must be expected. It would seem preferable to use lower tension steel contents than the limiting value allowed by ACI 318-71.

For example, if the compression steel ratio ρ' is 0.5 of the tension steel ratio ρ, in New Zealand it is recommended [10] that $\rho \leqslant 0.016$ when $f'_c = 3{,}600$ psi (25 MPa) and $\rho \leqslant 0.022$ when $f'_c = 5{,}800$ psi (40 MPa), with linear interpolation between for other concrete strengths. When $\rho'/\rho > 0.5$, higher ρ values than above can be used. These limits are given by formulae [10] based on analytical results [7].

Longitudinal Steel Content in Prestressed Concrete Beams

For prestressed concrete beams few codes give guidance for seismic design but recently the New Zealand Prestressed Concrete Institute has published recommendations [29] based mainly on work conducted at the University of Canterbury [21,30,23]. This research has shown that properly detailed prestressed concrete members will give satisfactory seismic resistance, although the lower hysteretic energy dissipation of a prestressed concrete member compared with a reinforced member of the same strength and initial stiffness will generally lead to a greater deformation response of the prestressed concrete member to a severe earthquake. Good confinement of concrete and a small neutral axis depth are the most important requirements for adequate curvature ductility. It is recommended that a/h should not exceed 0.2 in beams unless the very heavy confining steel typical of plastic hinge regions of columns is present, where a is the depth of the compressive rectangular stress block and h is the overall depth of the member. The presence of nonprestressed reinforcement in plastic hinge regions also effectively assists the ductility when acting as compression reinforcement and improves the hysteretic energy dissipation of the plastic hinge by "fattening" the hysteresis loops [23]. It is best to control curvature ductility in prestressed concrete beams by specifying a limiting neutral axis depth rather than a limiting steel content,because arrangements of tendons down the depth are often used and all the prestressing steel in the section is in tension but at different stress levels. Specifying a limiting a/h ratio gives a consistent value for the available ultimate curvature.

Concrete Confinement and Longitudinal Steel Support in Plastic Hinge Regions

The potential plastic hinge regions in beams are taken as the end 2h of the member, and if the critical section occurs away from the column face over a length 2h straddling the critical section, where h is the overall depth of the section. The spacing of stirrup ties in such regions of reinforced and prestressed concrete beams should not exceed 4 in (100 mm) or one-quarter of the effective depth of the beam (where the effective depth need not exceed 0.8h in the case of prestressed concrete beams) in order to ensure effective concrete confinement [31,23,49,50].

In reinforced concrete beams,with cyclic flexure to yield applied in each direction,a full depth crack may exist down the full depth of the section for much of the loading range due to residual plastic tensile strains in the steel (see Fig. 6). Also, the reinforcing bars may yield alternatively in tension and compression resulting in a lowering of the tangent modulus of the steel owing to the Bauschinger effect. This could lead to buckling of reinforcing bars in compression at lower load levels than expected. It is recommended therefore [7] that to prevent bar buckling in plastic hinge zones the spacing of stirrup ties surrounding the compression steel should not exceed six compression steel bar diameters, a spacing which is much smaller

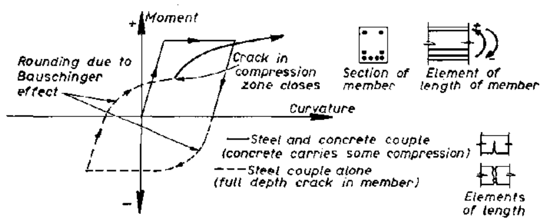


Fig. 6 Moment-Curvature Relationship for Doubly Reinforced Concrete Section With Reversed Flexure

than recommended in most current codes. This requirement also applies to stirrup ties around nonprestressed longitudinal bars in partially prestressed concrete beams. Also the tie force at yield provided by the stirrup tie should be at least one-sixteenth of the yield force of the longitudinal bar or bars it is to laterally restrain when spaced at 4 in (100 mm) centres.

Figure 7 shows the plastic hinge region of a reinforced concrete beam adjacent to a column face after the beam had been subjected to cyclic flexure well into the inelastic range [24]. The beam was of width 4.9 in (125 mm) and of effective depth 6.6 in (168 mm), and contained No. 4 (12.7 mm dia.) longitudinal steel bars as shown in the top and No. 2 (6.4 mm dia.) stirrup ties at 4 in (102 mm) centres (which is 0.6 ld). The compressed concrete is obviously inadequately confined, considering the depth of the beam, and the bars have buckled in compression. Had the stirrup ties been placed at

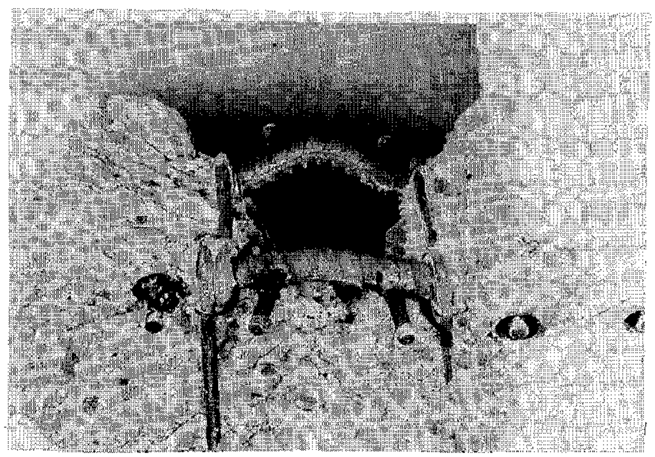


Fig. 7 Plastic Hinge Region of a Reinforced Concrete Beam After Cyclic Flexure in the Inelastic Range [24]

d/4 = 1.6 in (41 mm) centres, as is recommended above, the concrete confinement would have been more adequate. A 4 in (102 mm) spacing is more reasonable for large members where penetration of crushing between stirrup ties does not reduce the effective depth of concrete so significantly. The recommended spacing of not greater than six longitudinal bar diameters for stirrup ties to prevent bar buckling is 3 in (76 mm) for this beam and it is evident that the 4 in (102 mm) spacing actually provided was too great to prevent buckling.

Redistribution of Bending Moments Found from Elastic Frame Analysis

Since the plastic hinge regions are detailed for ductile behaviour it is also considered that in beams up to 30% redistribution of the absolute maximum moment derived for any of the load combinations is appropriate, provided the other moments are modified to satisfy the requirements of statics.

Shear Strength

The deterioration of the concrete due to the opening and closing of cracks in plastic hinge zones with cyclic bending moment decreases the concrete shear resisting mechanisms (aggregate interlock, dowel action and across the compression zone) [7]. In such regions only truss action of the stirrups should be relied on to carry shear, and where the shear force is high the full depth cracks should preferably be crossed by diagonal reinforcement. Examination of available test results for beams with cyclic flexure has indicated that where the cyclic shear force at the plastic hinge zone is such that the nominal stress there exceeds $3\sqrt{f'_c}$ psi ($0.25\sqrt{f'_c}$ MPa) a reduction in stiffness may occur in each load cycle due to shear and failure may be initiated by shear. The shear failure can occur by sliding along a full depth vertical crack in the plastic hinge zone. Thus it is recommended that the maximum nominal shear stress should not exceed $3\sqrt{f'_c}$ psi ($0.25\sqrt{f'_c}$ MPa) in each direction unless diagonal reinforcement exists in the web to cross the critical vertical cracks. Where the high shear force occurs in one direction only, the limiting nominal shear stress at which diagonal reinforcement is required can be increased.

In order to avoid shear failure the design shear force used needs to be calculated on the basis of the design gravity loads on the members and the likely overstrength moment capacity of the plastic hinges at the ends of the members. The plastic hinge moment capacities are calculated using a realistic steel strength, bearing in mind the likely excess of the actual strength over the specified value. For reinforced concrete it is likely that the actual yield strength will exceed the specified yield strength and that strain hardening will occur when developing the ultimate curvature. As a result, in New Zealand for steel with a specified f_y = 40 ksi = 275 MPa the use of a steel strength of $1.25f_y$ is recommended in such calculations; for steel with a specified f_y = 55 ksi = 380 MPa use of a steel strength of $1.40f_y$ is recommended due to the short yield plateau. For prestressed concrete the actual tensile strength of the steel should be used. Also, for beams cast monolithically with slabs, in negative moment regions account needs to be taken of the likely contribution of the slab steel to the flexural strength of the beam. For example, at interior columns where beams exist in both directions, the reinforcement within a distance of four slab thicknesses each side of the column could be included.

Required Further Research

It is evident that more testing of beams under cyclic loading is required to further examine and refine some of the above design recommendations which are still fairly subjective. Areas requiring more clarification are:

the maximum allowable longitudinal tension steel content, the possible advantages of using partially prestressed concrete beams, the transverse steel required to confine the concrete and prevent buckling of the longitudinal compression bars, the design of shear reinforcement in plastic hinge zones particularly the use of diagonal bars when the nominal shear stresses are high, and the contribution of slab reinforcement to the negative moment strength of beams at column faces.

COLUMNS IN FRAMES

Evaluation of Actions

The strong column - weak beam design concept aims at having plastic hinges form in the beams rather than in the columns. Some codes, for example the seismic provisions of ACI 318-71 [2] require that at beam-column connections the sum of the moment strengths of the columns should exceed the sum of the moment strengths of the beams along each principal plane at the connection. This requirement unfortunately will not prevent plastic hinges forming in the columns for three reasons:

(a) The actual beam steel strength at high curvatures will be higher than the specified yield strength and this strength will be further enhanced by strain hardening. Therefore the beam input moment may be considerably higher than that calculated using the specified yield strength.

(b) Nonlinear dynamic analyses have shown that in frames, due to higher mode effects, points of contraflexure may occur well away from the mid height of columns at various stages during an earthquake [18,32,35,7]. Thus bending moment distributions in columns such as in Fig. 8 are possible. Hence in Fig. 9, which shows a possible distribution of column bending moments, the beam input moments $M_{b1} + M_{b2}$ may have to be resisted almost entirely by one column section. In the extreme if the point of contraflexure lies outside the column height the strength of one column section needs to exceed $M_{b1} + M_{b2}$. This required column strength to prevent plastic hinges forming is much greater than the ACI 318-71 requirement.

(c) A general direction of seismic loading also causes a severe condition for the columns [33,34,7]. In design it is customary to consider seismic loading to act in the direction of the principal axes of the structure and in one direction at a time. However a general direction of severe seismic loading can cause yielding of the beams in both directions simultaneously. For example, for the symmetrical building shown in Fig. 10, if a displacement ductility factor of 4 is reached in direction 2 it only requires $\Delta_1 = \Delta_2/4$ to cause yielding in direction 1 as well, and this occurs when θ is only 14°. Thus yielding in the beams in both directions may occur simultaneously for much of the seismic loading. Biaxial bending will generally reduce the flexural strength of the column. Typically the flexural strength of a square column for bending about a diagonal may be 15% less than the flexural strength for uniaxial bending. Also, for a structure with beams of equal strength in each direction, the resultant beam moment input applied biaxially to the columns is $\sqrt{2}$ times the uniaxial beam moment input. Therefore concurrent earthquake loading may cause the columns to yield before the beams unless columns are strengthened to take this effect into account. Similarly, concurrent earthquake loading will induce higher shear forces in columns and joint cores when the beams yield in both directions than for loading in one direction only, and this higher shear force is to be resisted by sections loaded along a diagonal.

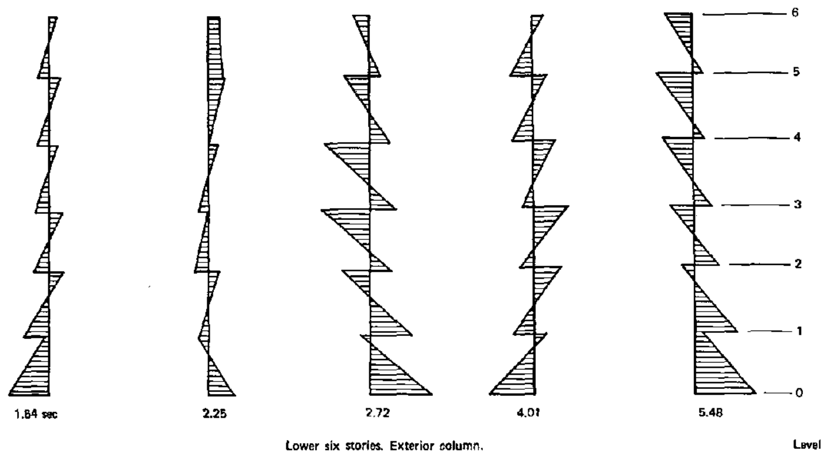


Fig. 8 Bending Moments in Lower Columns of a 12 Storey Frame Responding Nonlinearly to El Centro Earthquake 1940 N-S Component [32]

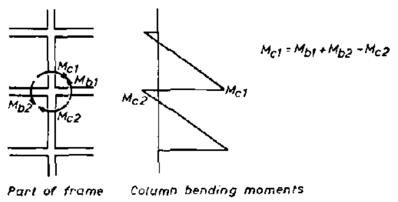


Fig. 9 Possible Column Bending Moments During Dynamic Response

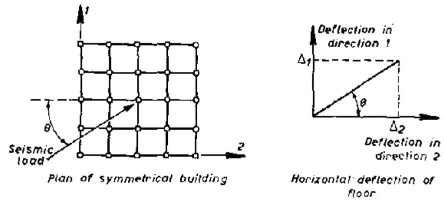


Fig. 10 General Direction of Earthquake Loading on Building

It is evident that column flexural strengths of rather greater than twice the ACI requirements would be needed if plastic hinges in columns are to be avoided. The difficulty of preventing plastic hinges from forming in columns is such that some column yielding must be considered to be inevitable. Note that yielding due to shift of the points of contraflexure in the columns due to higher mode effects will only occur at one end of the columns and therefore will not lead to a column sidesway mechanism in that storey. The degree of protection of columns against yielding is a debatable issue and needs to be approached on a probabilistic basis.

A design procedure developed by Paulay [11] is being recommended in New Zealand for determining column actions. This procedure is aimed at giving reasonable protection against column yielding. In the procedure the design uniaxial bending moment for the column, acting separately in each of the two principal directions of the structure, is given by

$$M_{col} = \phi_o \omega M_{code} \tag{6}$$

where M_{code} is the column moment at the beam centre line derived from the code loading and to be reduced as indicated by the moment gradient to give the moment at the beam face; ϕ_o is ratio of the overstrength flexural capacity of the beams as detailed to the beam moment capacity required by the code (for steel with f_y = 40 ksi = 275 MPa, if 1.25 is the overstrength factor for the steel and 0.9 is the capacity reduction factor used in beam design for flexure, ϕ_o is at least equal to 1.25/0.9 = 1.39); and ω allows for higher mode and concurrent loading effects, and ranges between 1.2 and 1.8 for one-way frames and between 1.5 and 1.9 for two-way frames, depending on the period of the building. The recommended values for ω were obtained from an assessment of the results of available nonlinear dynamic analyses. Note that in two-way frames the columns are designed for uniaxial bending only, since ω includes some moment enhancement to make allowance for the effect of biaxial bending. The design axial force P_{col} to be used with M_{col} in section design is calculated from the shear forces applied at the column faces by the gravity loads from the beams, and the moment induced shears from the beam plastic hinge moments in both directions acting at flexural overstrength, except that a reduction in the moment induced shears is allowed to take into account the probability that not all beam plastic hinges have reached their overstrength up the height of the frame. This reduction in moment induced shears is at least 0 to 30%, increasing from zero as the number of storeys increases. In this procedure it is also recommended that the column be designed for M_{col} and P_{col} using a capacity reduction factor ϕ of unity, since the effects of steel strength and moment patterns have been examined closely and the column sections will be detailed for ductility.

Concrete Confinement and Longitudinal Steel Support in Potential Plastic Hinge Regions

The possibility of yielding occurring at the column ends due to the effects discussed above makes it important to ensure that columns are capable of behaving in a ductile manner. Hence for reinforced and prestressed concrete columns adequate transverse steel in the form of hoops or spirals should be present at the potential plastic hinge regions at the column ends, to ensure ductile concrete behaviour and to prevent buckling of the longitudinal steel. The potential plastic hinge regions at the column ends can be taken as not smaller than the larger column section dimension or one-sixth of the clear height of the columns or 18 in [450 mm].

Code provisions for confining steel are at present based on rather arbitrary assumptions. For example, the amount of transverse reinforcement in the potential plastic hinge regions at the ends of columns recommended by the ACI [2] and SEAOC [36] Codes is based on preserving the axial load strength of the column after the cover concrete has spalled rather than aiming to achieve a particular curvature ductility factor for the column. The amount of rectangular hoops and ties specified is also based on the same criterion and assumes that rectangular hoops, because of their shape, are less efficient than circular spirals in confining the concrete. The philosophy of maintaining the axial load strength of columns after the spalling of cover concrete does not properly relate to the detailing requirements of adequate plastic rotation capacity of eccentrically loaded column sections. A more logical approach for the determination of the required content of transverse steel would be based on ensuring a satisfactory moment-curvature relationship and would include as variables the level of axial load on the column, the longitudinal steel ratio, the proportion of the column section confined, the stress-strain curve of the longitudinal steel, and the stress-strain curve of the confined concrete as a function of the amount of confining steel [7].

Moment-curvature analyses of column sections for monotonic flexure have been conducted using idealized stress-strain curves for steel and concrete [7, 37-40]. The stress-strain curve for the steel included the effect of strain hardening. The stress-strain curve for the concrete included the effect of the confining steel. Fig. 11 shows the assumed curve for concrete confined by rectangular hoops [27]. The slope of the falling branch is defined by the parameter Z which is a function of the concrete strength and the spacing and volume ratio of rectangular hoops. The value of Z decreases as the content of hoops increases. For concrete confined by circular spirals a curve with greater strength increase and ductility was used [40]. The moment-curvature curves were calculated using the idealized stress-strain curves, assuming that plane sections remain plane and satisfying the requirements of strain compatibility and equilibrium. The cover concrete was assumed to become ineffective at a compressive strain greater than 0.004. Fig. 12 shows a rectangular section with the idealized strain and stress distributions at a particular curvature. Fig. 13 shows the moment-curvature relationships obtained for a rectangular section with axial load held constant at $0.3f'_cA_g$, for a range of longitudinal steel ratios ρ_t and rectangular hoop contents. For the arrangement of hoops shown in Fig. 12, for this column section Z = 155 corresponds to No. 3 (9.5 mm dia.) hoops at 12 in (305 mm) centres and Z = 15 corresponds to No. 6 (19.1 mm) dia. hoops at 4 in (102 mm) centres. For this column section the content of special transverse steel recommended by the ACI [2] and SEAOC [36] codes is equivalent approximately to Z = 13.

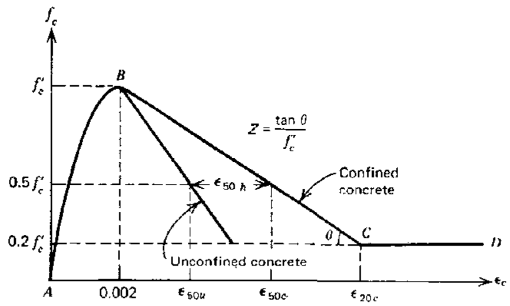


Fig. 11 Stress-Strain Curve for Concrete Confined by Rectangular Hoops [27]

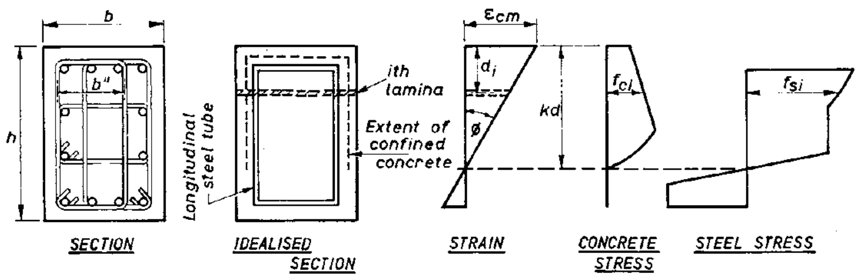


Fig. 12 Rectangular Reinforced Concrete Column Section With Idealized Strain and Stress Distributions

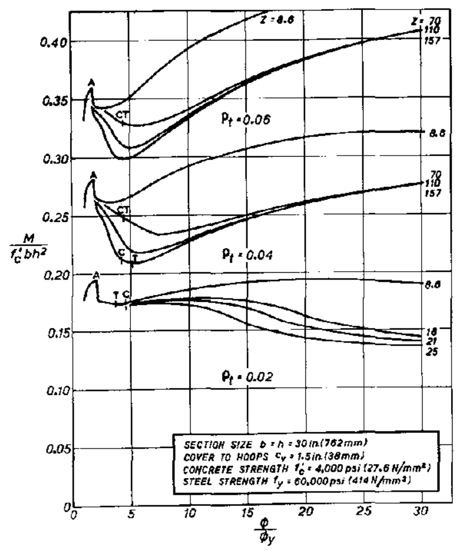


Fig. 13 Moment-Curvature Relationships for Reinforced Concrete Column Section With Compressive Load of $0.3f'_cA_g$ [38,7]

Moment-curvature analyses of columns conducted as above for monotonic flexure show a decrease in moment capacity when the cover concrete spalls, but providing adequate confining steel is present sections can maintain substantial moment with further plastic rotation. A difficulty with refined moment-curvature analyses of this type is that insufficient experimental data is available to accurately establish the stress-strain curve for confined concrete including the effect of overlapping hoops and hoops with supplementary cross ties. However the approximate analyses referred to above have shown that the equations for transverse steel content recommended in the 1973 SEAOC Code [36] are generally conservative for moderate axial load levels but not conservative at high load levels. Thus in New Zealand the SEAOC equations have been modified to take axial load level into account [12]. The axial load level is expressed as the ratio P_e/f'_cA_g, where P_e is the maximum design compressive load acting on the columns, f'_c is the specified compressive cylinder strength of the concrete, and A_g is the gross area of the column section. The recommended equations result in the following amounts of circular spiral steel being placed as a percentage of the amount recommended in the 1973 SEAOC Code:

P_e/f'_cA_g	0.1	0.2	0.3	0.4	0.5	0.6	0.7
% of SEAOC ρ_s	50	63	75	88	100	113	125

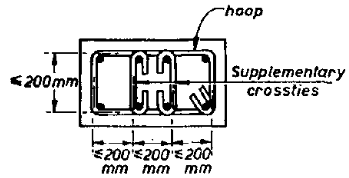
where ρ_s is the ratio of the volume of the spiral steel to the volume of concrete core. For rectangular hoop steel, with or without supplementary cross ties, the recommended equations result in the following amounts of transverse steel being placed as a percentage of the amount recommended in the 1973 SEAOC Code:

P_e/f'_cA_g	0.1	0.2	0.3	0.4	0.5	0.6
% of SEAOC A_{sh}	50	66	83	100	117	133

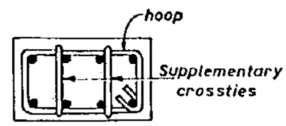
where A_{sh} is the total area of transverse steel crossing the section. The moment-curvature analyses indicated that use of the above recommended amounts of confining steel should result in curvatures being reached which generally are much greater than five times the yield curvature (defined as when the outermost tension bars first begin to yield) accompanied by a moment capacity which is generally not less than 80% of the moment capacity calculated at an extreme fibre concrete strain of 0.003, for columns with longitudinal steel ratio of 0.02 or greater, providing that the axial compression does not exceed either $0.7f'_cA_g$ for columns with circular spiral reinforcement or $0.6f'_cA_g$ for columns with rectangular hoop reinforcement. Also, experimental evidence [41] from reinforced concrete cantilever columns with circular spirals tested under cyclic flexure to displacement ductility factors of up to six have confirmed that the spiral steel content recommended above for axial compression of $0.1f'_cA_g$ should result in adequate ductility. However the moment-curvature analyses have shown that the curvature ductility available from very heavily loaded columns is doubtful even with large quantities of confining steel [37,38 39,40], and it is recommended that columns with circular spirals with $P_e > 0.7f'_cA_g$ and columns with rectangular hoops, with or without supplementary cross ties, with $P_e > 0.6f'_cA_g$ should not be used unless special studies show them to give adequate ductility. The greater limiting axial load for columns with circular spiral reinforcement compared with columns with rectangular hoop reinforcement is because it has been found from the analytical studies that the SEAOC specified spiral steel confines the concrete more effectively than the specified hoop steel. The moment-curvature analyses also demonstrated that use of high strength steel as longitudinal reinforcement in columns improves the performance of the columns at high curvature because the early strain hardening of that steel helps to compensate for the loss of moment capacity due to the reduction of contribution from the concrete. Also high longitudinal steel contents result in a smaller reduction in moment capacity at high curvatures.

Although derived for reinforced concrete columns, it is apparent that the above recommended quantities of transverse steel could also be used to confine prestressed concrete columns, in the absence of more accurate theory.

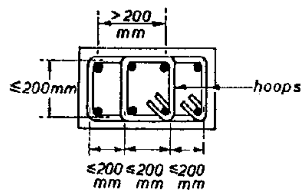
In most rectangular columns a single rectangular peripheral hoop will be insufficient to properly confine the concrete and laterally restrain the longitudinal steel against buckling. Therefore, an arrangement of hoops with or without supplementary cross ties will be necessary. Supplementary cross ties can only be expected to function effectively if fitted tightly around the bars, a rather difficult requirement in practice. Any gap left between the inside of the bend of the cross tie and the outside of the laterally supported bar will mean that outward expansion of the concrete needs to occur before the cross tie becomes fully effective, and thus some concrete confinement is lost. It would appear to be better to use a number of overlapping hoops rather than a single peripheral hoop and supplementary cross ties. Examples of some alternative details are shown in Fig. 14. The New Zealand recommendations require that supplementary cross ties and legs of hoops should not be spaced transversely more than either 8 in (200 mm) or one-quarter of the column section dimension perpendicular to the direction of the transverse steel. Also, the longitudinal column bars should not be spaced more than 8 in (200 mm) apart since they play an important role in assisting confinement. Supplementary cross ties can be anchored by bending around either a longitudinal bar or by bending around a peripheral hoop beside a longitudinal bar (see for example Fig. 14a and b). Note that it is not considered necessary for a supplementary cross tie to engage a hoop. That is, the concrete is confined by arching between hoops, supplementary cross ties and longitudinal bars. In a set of overlapping hoops it is preferable to have one peripheral hoop enclosing all column bars together with one or more hoops covering smaller areas of the column section. For example, the detail of Fig. 14c is preferred to that of Fig. 14d. This is because the longitudinal column bars are held more firmly in place during construction if they are all enclosed by one hoop.



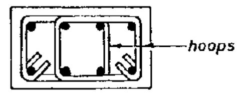
(a) Single hoop with supplementary cross ties bent around longitudinal bars



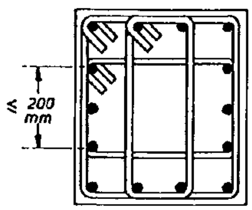
(b) Single hoop with supplementary cross ties bent around hoop



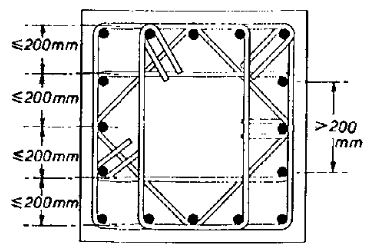
(c) Two overlapping hoops - preferred detail



(d) Two overlapping hoops - not preferred to (c)



(e) Three overlapping hoops



(f) Four overlapping hoops

Fig. 14 Some Details of Transverse Steel in Columns [12] (1 in = 25.4mm)

The spirals, or hoops with or without supplementary cross ties, are also necessary to provide lateral support to the longitudinal bars to prevent buckling. Hence, as for beams, the spacing between centres of spirals or hoop sets should not exceed six longitudinal bar diameters. However, not all longitudinal bars need to be laterally supported by a bend in a transverse hoop or cross tie. If bars or groups of bars which are laterally supported by bends in the same transverse hoop or cross tie are less than or equal to 8 in (200 mm) apart (see for example Fig. 14e) any bar or bundle of bars between them need not have effective lateral support from a bent transverse bar. Note also that the supplementary cross ties which are bent around hoops (see for example Fig. 14b) can be regarded as providing effective lateral support to the longitudinal bars beside them, since although they do not pass around those bars they effectively restrain the hoop beside the bar. Such supplementary cross ties should be secured to the adjacent longitudinal bars during construction. In large column sections the use of inclined ties (see for example Fig. 14f) helps to keep the centre of the section free from congestion of transverse steel thus allowing better access for concrete placement. The yield force of the hoop bar or supplementary cross tie providing lateral restraint should be at least one-sixteenth of the yield force of the longitudinal bar or bars it restrains.

Shear Strength

The design shear force for columns can be obtained from the moment gradient in the column. For small axial load levels, say when $P_e < 0.1f'_cA_g$, the shear carried by the concrete should be ignored in potential plastic hinge regions at the column ends. Strictly, the diagonal shear force resulting from biaxial bending in two-way frames due to concurrent seismic loading should be considered in design. The shear strength of rectangular column sections loaded along a diagonal has received little attention in the past. Tests have been conducted recently on four reinforced concrete members with a 16 in (406 mm) square section [42] subjected to uniaxial or diagonal shear force and flexure. Two arrangements of overlapping hoops were used. The members were designed to fail in shear and for convenience were tested in a horizontal position with no axial load applied. A member tested with diagonal shear is shown in Fig. 15. The photograph has been rotated 90° to show the member vertical as it would be in a frame. The difference between the diagonal shear strength and the uniaxial shear strength of identical specimens was zero for one pair and 3% for the other pair. This result is not surprising since although for diagonal shear the component of transverse bar forces in the direction of the shear force is smaller the diagonal tension crack has a greater projected length and therefore intercepts more transverse bars:these effects compensate each other.

However it is recommended in New Zealand that columns can be designed for uniaxial shear provided that the design shear force is calculated from the likely moment gradient associated with the enhanced uniaxial design moments discussed previously [11].

Required Further Research

A number of aspects regarding the seismic design of columns need further clarification. More information from nonlinear dynamic analyses of frames is required to obtain a better statistical basis for determining the design column actions which will give adequate protection against plastic hinging in columns, considering the effects of moment overstrength of beams, higher modes of vibration, and concurrent earthquake loading. More accurate confining steel provisions are required with more emphasis on flexural ductility, and including as variables the axial load level, type of steel, arrangement of transverse steel, effect of cyclic loading, etc. Such provisions can be derived analytically when accurate stress-strain curves for concrete confined by various arrangements of transverse steel become available. Further testing to determine more accurate tie requirements to prevent compression bar buckling is also required. Better systems for the mechanical splicing of longitudinal column bars suitable for seismic resistant structures would also be an advantage. There is also a scarcity of experimental results for the shear strength of rectangular columns loaded along a diagonal.

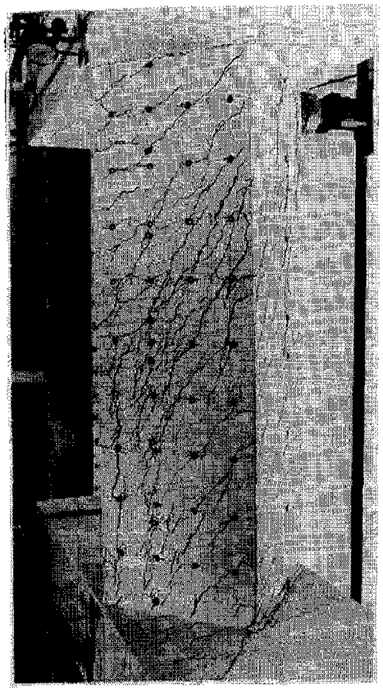


Fig. 15 Shear Failure of Reinforced Concrete Column With Shear Force Applied Along the Section Diagonal [42]

BEAM-COLUMN JOINTS

General

Ideally, the strength of a beam-column joint should be greater than the strength of the adjacent members since failure of the joint core may be non-ductile, the joint core is difficult to repair, and failure of the joint core could lead to the collapse of the column. In past years designers have tended not to give much attention to the detailing of beam-column joints. However, when subjected to seismic loading, beam-column joint cores can become critical regions of the structure, as is illustrated in Fig. 16. The performance of beam-column joints under pseudo-static seismic loading has been studied extensively in New Zealand in recent years [7, 43-54,21,23, 31]. These tests on reinforced and prestressed concrete beam-column sub-assemblages have indicated that when the plastic hinges form in the members adjacent to the connection the joint core may be subjected to extremely high shear forces and bond stresses. The design provisions for joint cores recommended by ACI 318-71 [2] have been questioned. In any case it would appear to be erroneous to base a design procedure for joint cores on test results obtained from members, as the ACI code has done.

Exterior Beam-Column Joints

Fig. 17 shows an external beam-column joint of a reinforced concrete frame and the associated forces and cracking. It is clear that the bond conditions for the longitudinal beam and column bars are unfavourable because: (a) large steel forces need to be transferred to the concrete over relatively short lengths of bar, (b) flexural and diagonal tension cracks are present which will alternate in direction during cyclic loading, and (c) bond deterioration will occur during cyclic loading. For example, if the outer column bars are near to yielding in compression above the core and are yielding in tension below the core, approximately twice the yield force of the bar needs to be transferred to the joint core by bond over the depth of the core. The extremely high bond stresses so induced, and the anchorage forces from the beam bars, can result in vertical splitting of the concrete along the outer column bars (see Fig. 18). Degradation of bond strength will also cause yielding of longitudinal bars to penetrate into the joint core, thus reducing the effective anchorage length and possibly resulting in slip of bars through the core. Therefore, in New Zealand it is recommended that at exterior beam-column joints in which

Fig. 16 Reinforced Concrete Beam-Column Joint Failure [43,44]

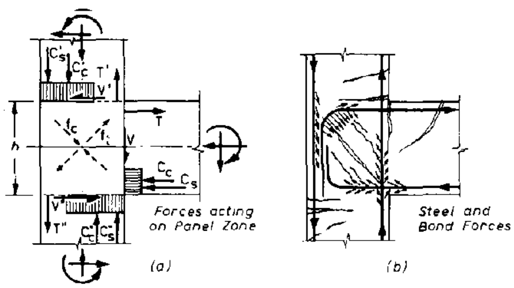


Fig. 17 Stress Resultants, Crack Pattern and Bond Forces at Reinforced Concrete Exterior Beam-Column Joint

plastic hinging occurs in the beam at the column face, the anchorage of beam steel should be considered to commence within the joint core at one-half the column depth or ten bar diameters, which ever is less, from the face of the column where the steel enters. An anchorage block, in the form of a beam stub at the far face of the column where the longitudinal beam bars can be anchored, has been shown [47,7] to result in considerable improvement in joint performance, and are being used by some designers in New Zealand (see Fig. 19). It is also recommended that the maximum diameter of longitudinal column bars should not exceed 1/20th of the beam depth for steel with f_y = 40 ksi = 275 MPa or 1/25th of the beam depth for steel with f_y = 55 ksi = 380 MPa.

The tests have also shown that when plastic hinging occurs in the beam adjacent to the column the ACI 318-71 [2] design approach for joint core shear results in adequate joint core shear strength in the first cycle of loading in the inelastic range, but that degradation of shear carried by the concrete occurs in subsequent inelastic cycles. It is recommended that the nominal shear stress v_c carried by the concrete shear resisting mechanisms in the joint core should only be taken into account if the compressive load on the column exceeds $0.1f'_cA_g$. The degradation of shear carried by the concrete occurs due to repeated opening and closing of diagonal tension cracks in alternating directions and full depth cracks in the beam which results in the beam compression being transferred into the joint core by bond. These two occurrences reduce the ability of the diagonal compression strut across the joint core to act as an effective shear resisting mechanism [7]. The critical diagonal tension crack has been observed to run from corner to corner in the joint core and the horizontal shear reinforcement should be designed by summing the shear reinforcement bar forces which cross that corner to corner crack. The ACI 318-71 assumption of 45° cracking is difficult to justify since the cracking will

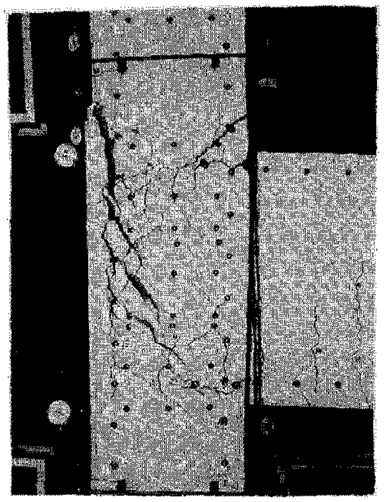


Fig. 18 Exterior Reinforced Concrete Beam-Column Joint Showing Splitting Cracks at Outer Longitudinal Column Bars [46]

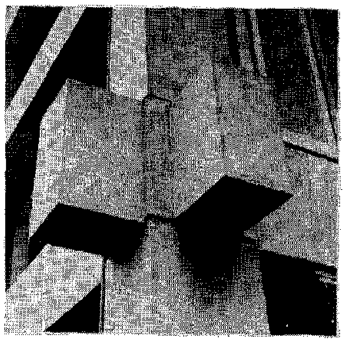


Fig. 19 Exterior Reinforced Concrete Beam-Column Joint With Anchorage Blocks

be parallel to the diagonal compression strut which runs from corner to corner. Hence, the design horizontal shear force in Fig. 17a is T - V', where T is the force in the beam bars enhanced for overstrength (for example, found using $1.25f_y$ for steel with a specified yield strength f_y = 40 ksi = 275 MPa) and V' is the column shear force. This design shear force should be resisted by the concrete if the compressive column load exceeds $0.1f'_cA_g$ and by the force in the horizontal shear reinforcement which crosses the corner to corner crack. Vertical shear reinforcement should also exist in the form of vertical column bars around the perimeter of the column section (spacing not to exceed 6 in (150 mm) and at least one intermediate bar between corners to be present). Such vertical bars are necessary to help transfer vertical shear forces. That is, four bar column should not be used. A procedure for the design of vertical shear reinforcement has been developed [14].

<u>Interior Beam-Column Joints</u>

Many of the points made regarding exterior beam-column joints apply to interior beam-column joints. Fig. 20 shows one of a series of interior beam-column joints being tested under pseudo-static cyclic loading. Fig. 21 shows a reinforced concrete joint which had been designed using the method of ACI 318-71 [2] and which failed in joint core shear,and slip of beam bars,after the first inelastic loading cycle. This can be contrasted with the behaviour of the partially prestressed concrete joint shown in Fig. 22 which had been designed according to the ACI 318-71 method but which had a prestressing tendon at mid-depth in the beam. The additional shear strength provided by this tendon crossing the diagonal tension cracks in the joint core enabled joint core shear failure to be prevented and allowed plastic hinging to occur in the beams. Slip of longitudinal beam steel through the joint core occurred in some of the tests [23,31,51,52]. When plastic hinging occurs in the beams at the column faces it is recommended that the maximum diameter of longitudinal beam reinforcing bars should not exceed

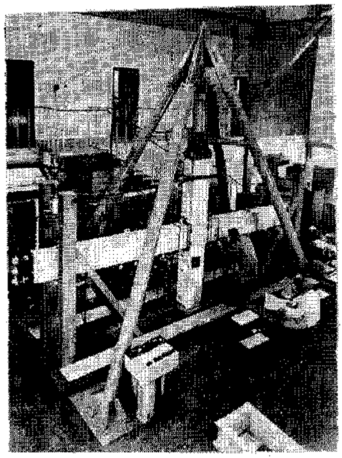


Fig. 20 Interior Beam-Column Joint During Testing [23,31]

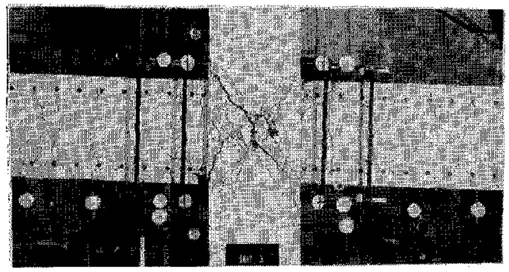


Fig. 21 Interior Reinforced Concrete Beam-Column Joint With Shear Failure in Joint Core [23,31]

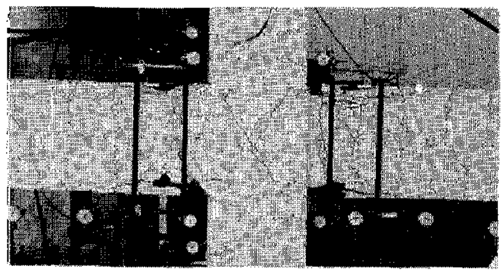
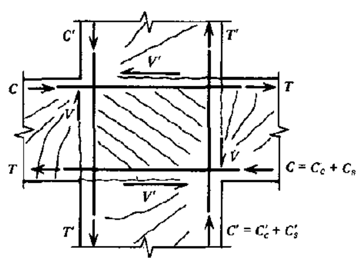


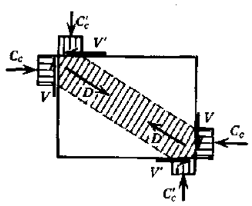
Fig. 22 Interior Partially Prestressed Beam-Column Joint With Flexural Failure in the Beams [23,31]

1/25th of the column depth for steel with f_y = 40 ksi = 275 MPa or 1/35th of the column depth for steel with f_y = 55 ksi = 380 MPa. The diameters of longitudinal column bars are limited as for exterior joints.

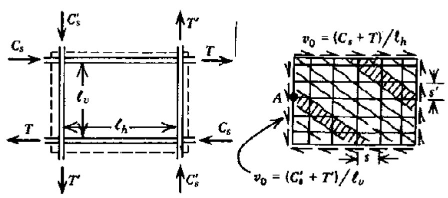
The degradation of joint core shear strength with cyclic loading occurs for the same reason as for exterior joints, namely repeated opening and closing of diagonal tension cracks, and full depth cracking in the beam at the column face, which leads to a reduction in the effectiveness of the concrete diagonal compressive strut. Fig. 23a illustrates the forces acting on a beam-column joint core. The forces entering the joint core are transferred across it by the diagonal compression strut (Fig. 23b) and by a truss mechanism involving diagonal tension and compression induced by the bond forces of the longitudinal bars (Fig. 23c). The shear carried by the concrete v_c arises mainly from the diagonal compression strut. When full depth cracking transfers most of the beam forces to the longitudinal steel the mechanism involving truss action becomes dominant and this mechanism requires the presence of both horizontal and vertical bars to carry the diagonal tension forces across the joint core. Hence the force to be carried by the horizontal shear reinforcement increases as cyclic loading proceeds and vertical steel crossing the joint core is needed to carry the vertical forces necessary to complete the truss mechanism.



(a) Forces on beam-column joint core



(b) Shear transfer by diagonal compression strut



(c) Shear transfer by truss mechanism

Fig. 23 Idealized Behaviour of Interior Beam-Column Joint Core

The design horizontal shear force in the notation of Fig. 23 is T + C - V', where the forces are calculated using steel stresses which include overstrength. A contribution to the shear strength from centrally placed prestressing tendons can be included. Therefore the design horizontal shear should be resisted by the concrete if the compressive column load exceeds $0.1f'_cA_g$, by the force in the prestressing tendons in the middle third of the beam depth if any, and by the total force in the horizontal shear reinforcement crossing the corner to corner crack. Vertical shear reinforcement should also exist as for exterior joints.

Biaxial Shear

Beam-column joint cores in two-way frame systems are subjected to high shear forces in the direction of the column section diagonal if the beams in both directions are yielding simultaneously. Fig. 24 shows the critical corner to corner crack when diagonal shear acts on an interior beam-column joint core. If the beams in the two directions are similar, the horizontal shear force acting along the diagonal of the joint core section is $\sqrt{2}$ times the uniaxial shear force. The corner to corner diagonal tension crack intersects the same number of shear reinforcement bars as for uniaxial shear, and if these shear bars are parallel to the sides of the section the diagonal component of bar force is $1/\sqrt{2}$ times that available to resist uniaxial shear. Hence design for biaxial shear for symmetrical two-way frames can lead to approximately double the quantity of shear reinforcement required for uniaxial shear design. This obviously is a serious problem. The confinement of the joint core from the transverse beams at right angles may aid the shear transfer by the concrete, and thus allow the shear carried by the concrete to be enhanced, but this may not be as effective as is assumed by some codes since full depth cracking at the column faces and damage at the plastic hinge sections of both sets of beams will reduce the

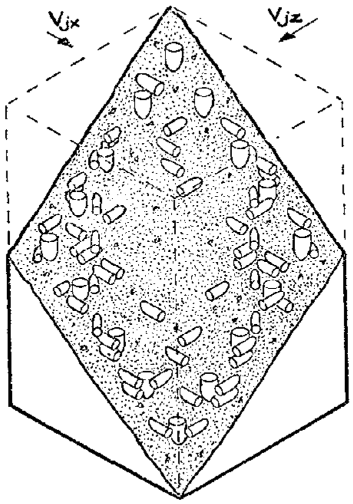


Fig. 24 Isometric View of Corner to Corner Crack Across Joint Core in Case of Diagonal Shear [14]

Fig. 25 Corner Joint of Model Reinforced Concrete Building Showing Diagonal Tension Cracks in Transverse Beam [55]

lateral confinement available to the joint core. A full-scale test specimen with biaxial joint shear is being tested in New Zealand this year to further investigate this case [51].

Torsion of Transverse Beams

Even with seismic loading acting in only one principal direction of two-way frames there may be secondary effects in the beams at right angles which could cause considerable damage. Large plastic hinge rotations in the beams at the column faces in the direction of seismic loading could induce large twists in the beams which enter the joint at right angles to the direction of seismic loading, owing to the presence of the floor slab cast monolithically with the beams. The imposed twist may cause diagonal tension cracking in the transverse beams which may affect their performance when the seismic loading acts in their direction. Fig. 25 shows a corner joint in a one fifth-scale six-storey reinforced concrete building [55] after testing with pseudo-static loading applied along one principal axis of the building. Cracking has developed across the corner of the slab and diagonal tension cracks have formed in the transverse beam. Such damage,particularly with intense cyclic loading, will add to the stiffness degradation of the building.

Plastic Hinging in Beams Away from Column Faces

The degradation of joint core shear strength, and the bond problems associated with longitudinal beam and column steel passing through the joint core, can be greatly reduced if yielding of longitudinal steel is forced to occur away from the faces of the joint core. This design concept of deliberately designing plastic hinges to form in the beams away from the columns is at present being investigated in New Zealand. Calculations have shown that the content of joint core shear reinforcement can be much reduced,and larger diameter longitudinal bars passing through the joint core can be tolerated, for this design situation. Plastic hinging can be forces away from column faces by suitable reinforcing details or by haunching the beams.

Required Further Research

The issues which need further research are: the actual mechanism of joint core shear resistance, the bar diameter criteria for anchorage as a function of more variables than the member depth, the vertical shear reinforcement necessary in joint cores, the contribution of prestressing to the joint core shear strength, the design for biaxial shear, possible non-conventional details for joint core design such as diagonal bars and bond plates, the effect of the presence of transverse beams, the effect of torsion induced in transverse beams, and design procedures to force beam plastic hinging away from the column faces.

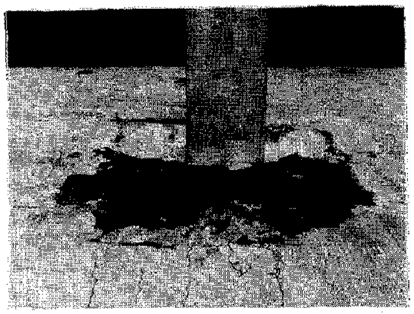
SLAB-COLUMN AND SLAB-WALL JOINTS

Slab-Column Joints

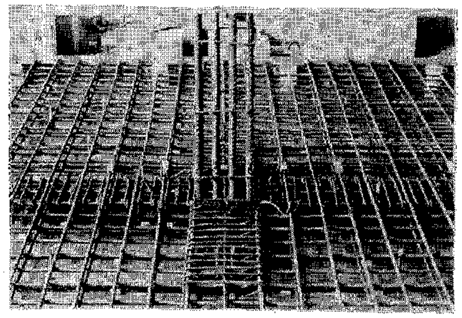
The transfer of unbalanced moment and shear at slab-column joints can be a critical aspect of the behaviour of flat plate structures. The reversals of unbalanced moment which occur during an earthquake could lead to a shear failure in the slab around the column due to degradation of the shear strength. It is not suggested that flat plate buildings should be used as seismic resistant structures without the presence of some frames or walls to stiffen the structure. Even with such stiffening elements present substantial unbalanced moment may need to be transferred at the slab-column connection, and these should be made adequately ductile. Some pseudo-static cyclic load tests have been conducted in New Zealand [56,57,58] on reinforced concrete slab-column joints with various arrangements of shear reinforcement in the slab. Fig. 26a shows a joint without shear reinforcement after loading to failure with shear and unbalanced moment. The final stage of shear failure occurs by the column punching through the slab at the critical face of the column and the top slab bars on that side of the column splitting off the slab top cover concrete. Structural steel shearheads were found to lead to an increase in strength and some ductility of the joint. The best detail, however, was the use of stirrup ties placed in the slab around those slab bars that pass through the column (see Fig. 26b). In addition to acting as shear and torsional reinforcement, the stirrup ties held the top and bottom slab steel together and prevented the column from punching through the slab at the critical face thus suppressing a brittle failure. Fig. 26c shows a joint with stirrup ties placed as in Fig. 26b at failure and the most evident sign of damage is now due to torsion in the slab near the side faces of the column. The use of stirrup ties placed around the slab bars passing through the column results in a substantial increase in ductility of the joint. The strength of joints reinforced in this manner can be determined using an approach which sums the flexural, shear and torsional strength contributions around the column using a beam analogy [56,58].

Slab-Wall Joints

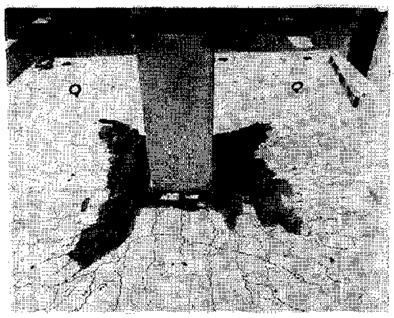
A series of tests on reinforced concrete slab-wall joints has recently been completed in New Zealand [20]. Various arrangements of shear reinforcement were used in the slabs. The tests have not yet been fully reported. It was shown to be difficult to prevent shear failure at the critical toes



(a) Joint without shear reinforcement



(b) Stirrup ties as shear reinforcement



(c) Joint with stirrup ties as shear reinforcement

Fig. 26 Reinforced Concrete Slab-Column Joints [56,57]

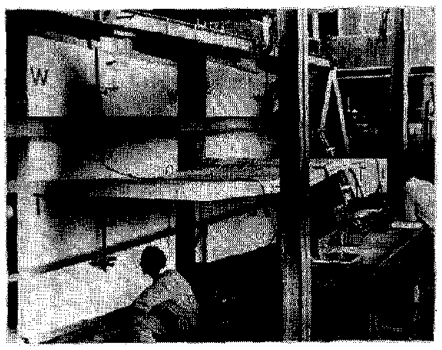


Fig. 27 Reinforced Concrete Slab-Wall Specimen Under Test [20]

of the wall but nevertheless yield lines developed across the slab and allowed the full flexural strength of the slab to be developed. Fig. 27 shows a specimen during pseudo-static cyclic load tests.

Required Further Research

Further testing of slab-column and slab-wall joints is necessary, particularly for prestressed concrete slabs, in order to extend the knowledge of detailing for ductility. Most existing slab-column tests have been conducted on single column specimens in which loading has been applied which attempts to simulate the conditions at the joint of a multipanel structure. Testing of multipanel specimens is also necessary to check whether any unexpected effects may be caused by the actual distribution of actions around the joint.

COMPLETE FRAMED STRUCTURES

General

Two one-fifth scale reinforced concrete model structures, one bay by one bay wide and six-storeys high, have been tested [55,7]. One model was tested mainly under pseudo-static cyclic loading and the other was tested dynamically on a shaking table. The models had been designed for New Zealand code seismic loading and detailed to the requirements of ACI 318-71 [2] for ductile frames in seismic zones. A model under static loading is shown in Fig. 28. The static cyclic horizontal loading resulted in considerable stiffness degradation, mainly due to cracking and to slip of bars in anchorage zones. The dynamically loaded model suffered little damage when subjected to the El Centro 1940 N-S earthquake. The nonlinear dynamic analysis of this model using Sharpe's program [18] demonstrated that an accurate estimation of the stiffness and viscous damping of the structure was essential for the accurate prediction of its response to dynamic loading. Using those stiffness and viscous damping values which gave the best fit between the predicted and measured responses of the structure to the strong motion portion of the El Centro 1940 N-S earthquake, the displacement response could be accurately predicted but the degree of accuracy decreased as stiffness degradation set in. This emphasizes that nonlinear dynamic analyses require realistic input parameters if accurate response predictions are to be achieved.

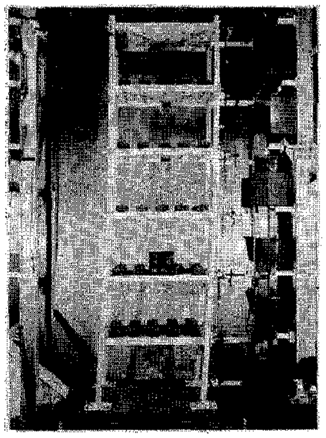


Fig. 28 Model Reinforced Concrete Building Structure Under Pseudo-Static Cyclic Loading [55,7]

Further Research

A good deal of research is required involving shaking table tests of complete structures for analysis, to give confidence in the use of available computer programs for nonlinear dynamic analyses, to check the accuracy of idealizations made in such analyses, and to improve such idealizations if necessary.

SHEAR WALLS

General

A considerable amount of research has been conducted in New Zealand in recent years on the behaviour of shear walls under pseudo-static seismic loading. This research has shown that properly detailed shear walls will provide adequate strength and ductility in buildings. Hence suspicions that all shear walls will fail in a brittle manner are groundless providing reasonable design precautions are taken. Reinforced concrete shear walls provide an attractive means of seismic resistance, helping to reduce problems such as column yielding, beam-column joint detailing, and instability due to drift. Their stiffness also enables much non-structural damage during a severe earthquake to be minimized.

Cantilever Shear Walls

Cantilever shear walls are in effect lightly loaded cantilever columns with narrow cross sections. The longitudinal reinforcement content is small and therefore they can be expected to behave in a ductile manner, providing lateral instability of the compression flange does not occur and that shear failure is prevented [7]. For tall cantilever walls higher modes of vibration can effect the distribution of shear force when the wall is loaded to flexural capacity [59]. The design shear force at the flexural capacity of the wall can be considerably higher than that calculated using the code distribution of static horizontal loading, and it is recommended that the design shear force should be found by factoring up the code shear at the flexural capacity by some 1.0 to 1.8 times, depending on the number of storeys amd the importance of the structure [1,59]. An experimental study on model squat shear walls has been conducted [60,66,7]. The walls were square (height to width ratio of unity), and the horizontal shear force was distributed along the top edge of the wall and applied cyclically. The tests showed that if a ductile (flexural) failure mechanism is required in a low rise shear wall the nominal shear stress associated with the flexural capacity of the wall must be small, say less than $6\sqrt{f'_c}$ psi ($0.5\sqrt{f'_c}$ MPa). Also no reliance should be placed on the contribution of the concrete to the shear strength; that is, all the shear force should be carried by the web reinforcement.

Coupled Shear Walls

Many shear walls contain vertical rows of openings, and the walls each side of the openings are connected by short deep beams. Extensive studies of the behaviour of coupling beams has been made in New Zealand, for example [61-72, 7]. When the wall is subjected to seismic loading the coupling beams are subjected to flexure and shear and because of their small span/depth ratio shear deformations of these beams may become very significant. A laminar analysis can be used to find the elasto-plastic response of coupled shear walls under monotonic loading, and so enable an assessment to be made of the ductility requirements of the coupling beams to achieve a given displacement ductility factor [61,62,7]. The experimental investigations conducted were to determine whether the required ductility can be achieved in deep coupling beams. Fig. 29 shows the test rig used to provide pseudo-static cyclic flexure and shear at the ends of a coupling beam to simulate conditions in a coupled shear wall during seismic loading. It was found [61-64,7] that for coupling beams with clear span/depth ratios less than about 2 the diagonal tension cracking caused a radical redistribution of the tensile forces and the whole length of the longitudinal bars, top and bottom, in the beams was in tension. Therefore no increase in ductility was available through compression steel, since the concrete carried all the compression. For clear span/depth ratios less than about 2, after high intensity load reversals, the attainable flexural strength was only about 85% of that predicted by conventional flexural strength theory. The diagonal tension cracking in alternating directions, shown in Fig. 30, reduced the capacity of the concrete to carry shear and eventually transferred all the shear to the stirrups. Therefore it is important to provide shear reinforcement to carry all the shear force at the beam flexural capacity. The stiffness of the coupling beams degraded significantly with cyclic loading and shear deformations were greater than flexural deformations. The ultimate failure for members adequately reinforced for shear was due either to crushing of concrete, or to shear slip along a vertical crack due to breakdown of aggregate interlock and the opening of the crack due to yielding of the longitudinal steel. Fig. 30 illustrates a sliding shear failure. Vertical stirrups cannot effectively prevent this type of shear failure and if conventional reinforcing details are used the nominal shear stress should be limited to ensure that sliding shear failure does not occur.

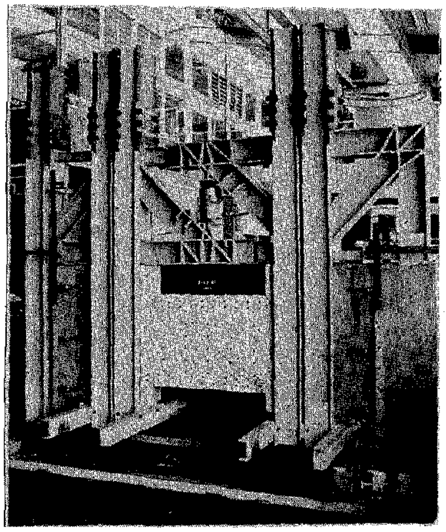


Fig. 29 Test on Reinforced Concrete Coupling Beam [61]

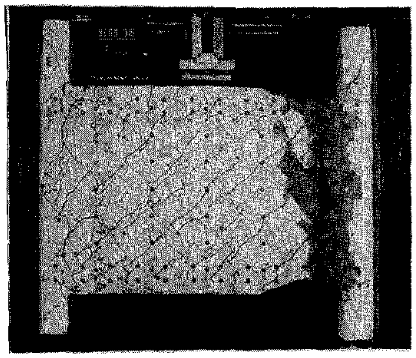


Fig. 30 Diagonal Tension Cracks and Sliding Shear Failure in Reinforced Concrete Coupling Beam [61]

Further experimental work has shown that the ductility and useful strength of coupling beams can be considerably improved if, instead of the conventional arrangement of longitudinal flexural steel and vertical stirrups, the principal reinforcement is placed diagonally in the beam [64-72,7]. Fig. 31 shows a possible arrangement of diagonal reinforcement. For such a beam the applied moments and shear are resisted by the internal diagonal compression and diagonal tension forces. When a full depth open crack exists after cyclic loading the diagonal steel carries both the moment and flexure without assistance necessary from the concrete other than stabilizing the compression bars against buckling. Diagonally reinforced coupling beams have been shown to have excellent characteristics, and the hysteresis loops have almost the stability of a steel member. Strength degradation only occurs if buckling of compression bars commences [64-72,7].

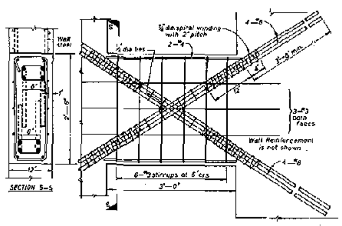


Fig. 31 Diagonal Reinforcement in Coupling Beam [7]

Two one-quarter scale reinforced concrete coupled shear walls have been tested under pseudo-static cyclic loading to verify behaviour of coupling beams in a complete structure during seismic loading [67-72,7]. Fig. 32 shows a model with diagonally reinforced coupling beams under test (the wall was tested horizontally for convenience) and also a model with conventionally reinforced coupling beams standing upright after testing. Sliding shear failure eventually occurred in all beams

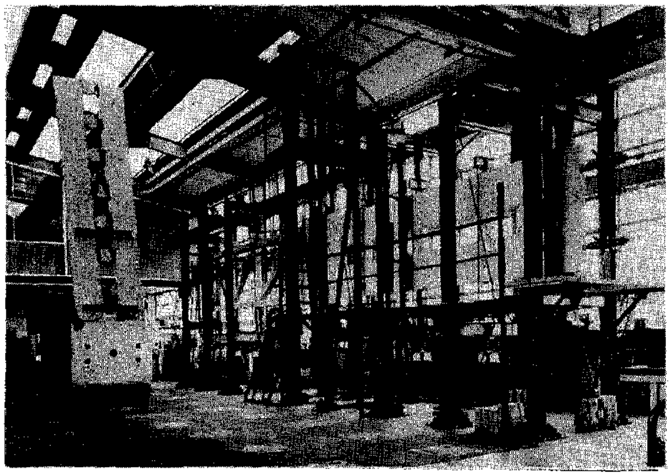


Fig. 32 Reinforced Concrete Coupled Shear Wall Models [69,70,72]

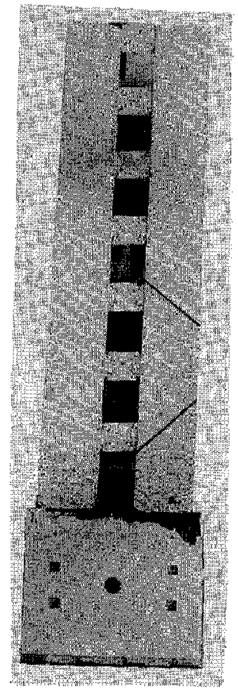


Fig. 33 Reinforced Concrete Coupled Shear Wall Model With Diagonally Reinforced Beams After Testing [69,70,72]

of the conventionally reinforced model. Fig. 33 shows the model with diagonal reinforcement after testing and it is evident that sliding shear failure of the coupling beams has not occurred. Both models showed significant ductility, but as expected the model with diagonal reinforcement showed less damage and more hysteretic damping ability. Diagonally reinforced coupling beams are now becoming commonly used by designers in New Zealand (see Fig. 34).

Shear Wall - Frame Interaction

Tests on two quarter-scale reinforced concrete shear wall - frame models under pseudo-static cyclic loading have recently been completed in New Zealand [73,74]. Fig. 35 shows one of the models on its side after testing. In the first model which had conventionally reinforced beams a sliding shear failure eventually occurred in all the plastic hinges near the wall, even though the nominal shear stress in the beams at the development of flexural capacity was quite moderate, being approximately 190 psi (1.3 MPa). In the second model the beams were reinforced diagonally in the plastic hinge regions and behaved very satisfactorily. Thus although adequate ductility can be achieved by conventional detailing, the diagonally reinforced members showed more stable hysteresis loops and less stiffness degradation.

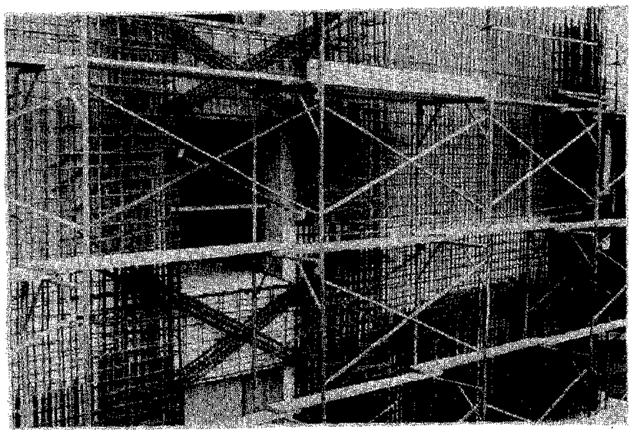


Fig. 34 Diagonal Reinforcement for Coupling Beams of Shear Walls During Construction

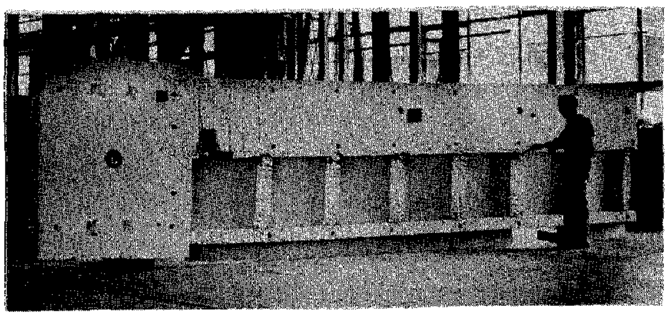


Fig. 35 Reinforced Concrete Shear Wall - Frame Model After Testing [73,74]

Construction Joints

Sliding movements are sometimes observed along horizontal construction joints in shear walls. Shear transfer by aggregate interlock (shear friction) along preformed cracks has been examined by monotonic and cyclic load tests [75,76,7]. Specimens have also been tested to study the contribution of dowel action, surface preparation, and reinforcing content to the shear strength of construction joints under monotonic and cyclic loading [77, 78,7]. Fig. 36 shows a test specimen after shear loading along a horizontal construction joint. It was found that adequately reinforced horizontal construction joints with a clean and rough surface, to which freshly placed concrete will bond, can develop an interface shear strength which is equal to or larger than the shear strength of the remainder of the wall. For design purposes dowel action should be neglected, since it is only developed at large slips. Use of an apparent coefficient of friction of unity with the shear friction provisions of ACI 318-71 should give a sufficiently conservative procedure in seismic design.

Required Further Research

Aspects of shear wall behaviour which require further investigation are: the efficiency of various shapes of wall cross section and the necessity for flanges or columns at the edges of walls; the criteria for lateral instability of the compressed edges of walls; the possible use of vertical prestressing tendons in walls, with nonprestressed steel only in potential plastic hinge regions if it is present at all, in order to avoid lapping reinforcing bars; the transverse steel details to confine the concrete and to prevent buckling of compression steel; diagonal reinforcement in members; and the reason why some designers in overseas countries consider it necessary to bury structural steel frames in reinforced concrete shear walls.

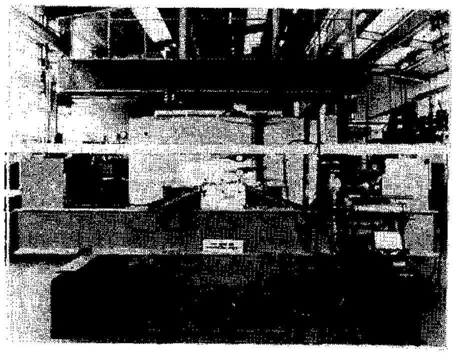


Fig. 36 Horizontal Construction Joint in Test Frame After Testing [77,78]

FULL SCALE SHAKING TESTS

Some small amplitude steady state vibration tests have been conducted in New Zealand on full scale reinforced concrete buildings which were excited in the elastic range by vibration apparatus clamped to the structure [79,80]. Such tests give valuable information concerning the dynamic characteristics of buildings and foundation compliance in the elastic range of response.

BASE ISOLATION OF BUILDINGS

A range of mechanical devices which act as hysteretic dampers have been investigated at the Physics and Engineering Laboratory of the Department of Scientific and Industrial Research, New Zealand. These energy dissipation devices may take the form of steel elements which bend, roll, or twist, lead extrusion or lead shear devices [81]. Some of these devices are suitable for insertion between the foundations and the structure of buildings to form base isolation systems. A recent study [82] using nonlinear dynamic analysis has demonstrated that base isolation is most efficiently employed in short to intermediate period structures. Seismic forces in the structure are decreased and hence ductility requirements are reduced. This method of protection against seismic loading holds much promise. It is a practical approach to design now, and no doubt will be more used when further detailed studies have been completed.

CONCLUSIONS

New Zealand has been active in recent years in updating its design codes. The code for general structural design and design loadings for buildings, which contains general provisions for seismic design, is now published. The code containing detailed provisions for concrete design, which is almost complete and is in draft form at present, will result in a high standard of detailing of reinforced and prestressed concrete structures for earthquake resistance. The concrete design code is based on current American practice for seismic design with additional provisions based on the findings of recent research. This code will be more extensive than previous editions but this is seen to be necessary. The capacity design procedures recommended are aimed at ensuring ductile behaviour of the structure and minimizing strength and stiffness degradation during severe earthquakes. Reinforced concrete is the dominant building material used in New Zealand. Prestressed concrete is now accepted for seismic resistant construction and this will be the first occasion that seismic provisions for prestressed concrete will be included in a New Zealand code.

A considerable amount of research and development into the design of earthquake resistant reinforced and prestressed concrete frames and reinforced concrete shear walls has been conducted in New Zealand in recent years. This work has been both analytical and experimental involving model and full-scale structural elements, subassemblages and complete structures. Particular emphasis has been given to material behaviour, detailing of plastic hinge regions, beam-column joint detailing, column protection from seismic actions, and shear wall detailing. The design profession has taken a lively interest in this research and development. An excellent interchange of views has taken place and agreement has been reached on seismic design procedures as a result of conferences and meetings. The New Zealand National Society for Earthquake Engineering has taken a leading role in maintaining excellent communication between research workers and designers.

An inevitable result of research and in-depth investigations is a crop of further problems. Although it is felt that a high standard of detailing has already been achieved, further research is necessary to improve and refine existing procedures.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
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DESIGN EARTHQUAKES - UNCERTAINTIES IN GROUND MOTION INPUT AND THEIR EFFECTS ON BUILDING CONSTRUCTION

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INTRODUCTION

The state-of-the-art and the state-of-the-practice in both the selection of and the uncertainty within ground motion input and the way the input are used vary substantially. This difference is so great that the requirements of this paper may be best served by separate discussions of the state-of-the-art as it is currently perceived and the means by which ground motions are presently considered in the vast majority of designs. While both topics are covered in an abbreviated form it is hoped that the list of references will serve as a guide for the interested reader.

CURRENT PRACTICE

The sole mandatory requirement for seismic design of structures at the present time is that the design be shown to satisfy the appropriate building code. The requirements of the particular code may be different but proof that they have been met must be demonstrated to building officials. This simple requirement has led to two divergent and counter-productive tendencies. The first comes from the designer who is often pressured by the architect and building owner to produce a design which satisfies the legal requirements but does not provide sensible seismic protection. The code is looked on in such a situation as an adversary instead of a guide. The second response comes from those who recognize that some designers do look at the code in this way and recommend with each code revision that design requirements should be increased. We now have a situation where schools in California built since the passage of the Field Act in 1933 and have a splendid performance record during recent destructive earthquakes do not satisfy the current codes.

Those structures which suffered the most distress in the San Fernando Earthquake were frequently found to satisfy the code requirements while having deficiencies in continuity etc. which cannot be remedied by code modification alone, especially the simple expedient of requiring higher forces. Higher force requirements will result in stiffer structures. As many of the major problems in earthquakes are produced by displacements, designs which produce stiffer structures may be self defeating, especially for relatively brittle structural materials such as reinforced concrete.

The 1974 Edition of the Recommended Lateral Force Requirements and Commentary of the Structural Engineers Association of California saw the first United States use of a factor relating different design force levels to different soil profiles. This factor which can increase the lateral force requirement by a factor of up to 1.5 requires the computation of the period of the structure and the characteristic period of the soil profile. In order to simplify the code procedures and recognize that ground motions cannot be categorized by any single parameter the provisions being developed by the Applied Technology Council [3] suggest a different method. For this procedure the ground motion is represented by two quantities A_a (the effective peak acceleration) and A_v (the effective peak velocity-related acceleration). For most parts of the country the values for A_a and A_v will be equal. In areas at moderate distances from the major seismic source zones the value of A_v will be larger than A_a. This difference is produced by the slower attenuation of ground velocity than ground acceleration with distance from the seismic source. The maps for A_a and A_v in contour form are shown in Figures 1 and 2. The contours show the quantities A_a and A_v which have approximately 80 to 95 percent probability of not being exceeded in 50 years. This probably represents the most significant change being contemplated, recognition of both the probable size and frequency of earthquake occurrence. For locations inside the maximum or minimum values a constant value equal to the maximum or minimum value is assumed. For locations between contours the values should be obtained by interpolation. If the use of maps based on Figure 1 and 2 are adopted in future codes they are expected to be in the form of zone maps with zone boundaries based on political jurisdictions, using county borders as the boundaries. In highly seismic regions such as California where counties pass through several contours some additional local subdivision would be advisable.

The adaptation of contour maps to zone maps based on political subdivisions is an example of the difference that exists and will continue to exist between the state-of-the-art and the state-of-the-practice. While contours are clearly the more preferable technique they are unacceptable to building officials who must adopt and then enforce code provision, so a compromise had to be made.

The code provisions for lateral force would be obtained using the following coefficient C_s with the weight of the structure

$$C_s = \frac{1.2A_vS}{RT_a^{2/3}}$$

where S is the soil factor as given in Table 1 below.

T_a is a simplified approximation to the fundamental building period for use in defining base shear

R is a response modification factor based primarily on ductility and damping considerations

TABLE 1

Soil Profile Coefficient

	Soil Profile Type		
	A	B	C
S	1.0	1.2	1.5

The three soil profile factors defined as follows:

Soil Profile Type A is a profile with:

1. Rock of any characteristic, either shale-like or crystalline in nature. Such material may be characterized by a shear wave velocity greater than 2,500 feet per second, or

2. Stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

Soil Profile Type B is a profile with deep cohesionless or stiff clay conditions, including sites where the soil depth exceeds 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

Soil Profile Type C is a profile with soft- to medium-stiff clays and sands, characterized by 30 feet or more of soft- to medium-stiff clay with or without intervening layers of sand or other cohesionless soils.

The maximum value of C_s is controlled by the quantity of A_a in the following way. The value of C_s need not exceed 2.5 A_a/R for Types A, B and C soils except where A_a is equal to 0.3 or greater. In that case C_s need not exceed 2.0 A_a/R for Type C soils. Examples of the lateral force coefficient curves given by this relationship are shown on Figure 3. The similarity of the form of these curves to spectral averages obtained by Seed et al [38] can be seen by comparison with Figure 4. The simpler concept describes the Lateral Force Coefficient by including site conditions directly and removes the requirement that T_s be computed.

The most significant change of all is the formal recognition that ground shaking larger than that recommended for design has a small but finite probability of being exceeded. Although previous code commentaries did not guarantee successful performance if the requirements were conformed to it was implied that they would be resisting major earthquakes. For example, the commentary to the Structural Engineers of California Recommended Lateral Force Requirements discusses resisting major earthquakes "of the intensity of severity of the strongest experienced." The recognition of a finite exceedance probability allows the treatment of earthquakes in a manner similar to other natural phenomena such as winds and floods. It also requires the careful consideration of the level of uncertainty in the choice of ground motion. Some aspects of this uncertainty are covered in a later section of this paper.

It is not possible to review in this paper different approaches to ground motion evaluation used in codes of countries other then the United States. Although approaches vary all codes are for regulatory purposes only. It is a simple but true observation that good design, including seismic design is impossible to achieve by legislation alone. Special structures currently use more sophisticated design procedures and require definition of design motions in more detail. This definition is usually done in a deterministic way either as a design response spectra, a design time history or both. Little consideration is given, once these deterministic parameters have been selected, to any possible design considerations that might be altered if the bases of the choice were more widely appreciated. Much judgement is required in the selection of design motions and different individuals will judge different levels of conservatism for design as being adequate. The building code or its equivalent will set the minimum standard. The question which remains to be answered is: how can a procedure be established to provide guidelines that might lead to a repeatable and reliable means of selecting ground motion?

STATE-OF-THE-ART

Interest has grown in the direct application of strong motion records to the design process since the first accelerograph records were obtained in the 1930's. Since that time the number of strong motion records has increased almost exponentially. The February 9, 1971 San Fernando earthquake provided the first opportunity to directly examine a large set of records from a single event to find possible effects related to soil profiles [13,17], location with respect to upthrown or downthrown block [2], etc. By the use of statistical procedures it has been possible to estimate not only the most probable values of the principal seismic parameters but also the amount of variability of these parameters. As these ground motion and spectral parameters are of direct use and interest in reinforced concrete design they will be addressed in moderate detail.

Before describing specific ground motion quantities and effects the probability of occurrence of the design motions should be considered together with the option of one or more design levels representing different loading conditions. The most familiar dual level requirement is that of the Nuclear Regulatory Commission in the United States. The upper of the two levels, the Safe Shutdown Earthquake, is represented as the maximum earthquake which a structure is likely to ever experience based on the geology and tectonics of the region. The lower level can be decided on purely economic grounds but is usually not. The Nuclear Regulatory Commission requirements state that the plant must be closed down for detailed inspection after the lower level motion has been exceeded. Until recently the motion levels for the lower level event were established as one-half of the safe shutdown earthquake. Probabilistic procedures are now accepted as a means of justifying a lower value.

While it had been thought that the safe shutdown earthquake represented close to the maximum possible event at the site, careful probabilistic and seismologic studies have suggested that the safe shutdown event motion levels have an annual probability of exceedance of 1×10^{-4} to 1×10^{-5}.

Probabilistic techniques are well described in the literature [4,6,7,8, 12,14,20,29] and other investigations [13,18,19,21,42] have studied the seismologic and geologic parameters which must be included in any probabilistic evaluation of design ground motions. Algermissen and Perkins [1] have produced a map of the contiguous states showing contour levels of peak acceleration in rock with a 90 percent probability of not being exceeded in 50 years. Their map was the starting point for the development of the ATC-3 acceleration map shown on Figure 1. There are several important points regarding the Algermissen-Perkins map which should be mentioned. They used in their study attenuation relationships published by Schnabel and Seed [37]. Unfortunately the Schnabel-Seed relationships do not represent mean values and their degree of uncertainty is not known so incorporation of this important factor into the work of Algermissen and Perkins will present some difficulties. The seismic source zones chosen by Algermissen and Perkins represent their best estimates based on available information. There are additional reasons why the Applied Technology Maps differ from those of Algermissen and Perkins. These are discussed by Donovan et al [15].

PEAK ACCELERATION

Peak acceleration has been the most widely used single strong motion parameter in studies of earthquake behavior. Its only advantage has been its ready availability especially as obtained from a paper or film record from a strong motion instrument. The lateral force coefficient on a structure is taken as some portion of its weight so the lateral force can be related to the acceleration of gravity. It should not and cannot be related to peak instrumental accelerations. Although conclusions to complex studies have been based on comparison of observed and computed peak accelerations peak acceleration is a notoriously inconsistent and widely varying parameter. The range that this variation can sometimes cover is shown on Figure 5 where data points obtained from strong motion instrumentation during a small, magnitude 5.5 earthquake which occurred near Ferndale, California on June 7, 1975, are compared with estimates of peak acceleration using different relationships. Some of the values on Figure 5 lie outside 2 standard deviations from the mean values upon which the Donovan and Esteva curves are based. In the near field recent work by Hanks and Johnson [26] has suggested that for magnitudes above 4.5 peak acceleration may have no relationship to either the earthquake size or the true severity of ground shaking on structures.

With such limitations why is acceleration still in use as a design ground motion parameter? The use of an acceleration term is retained as a scaling term against which all other design ground motion terms are related. In this usage the instrumental peak acceleration is not used. The Applied Technology Council Study adopted a term called Effective Peak Acceleration (EPA) which is defined in the following way. "For a specified actual ground motion of normal duration, EPA and EPV (effective peak velocity) can be determined as illustrated in [Figure 6]. The 5 percent damped spectrum for the actual motion is drawn and fitted by straight lines between the periods mentioned above (see figure). The ordinates of the smoothed spectrum are then divided by 2.5 to obtain the EPA and EPV. The EPA and EPV thus obtained are related to peak ground acceleration and peak ground velocity but are *not necessarily the same as or even proportional to peak acceleration and velocity.*"

The EPA value therefore is similar in magnitude to the average instrumental maximum value but should not be expected to be equal to any individual value. This same form of spectral averaging using normalization of response spectra to produce design motions has been performed by many investigators [5,23,24,25,32,33,34,38] since the first averages of Housner [27].

The uncertainty in the data set has been included in the derivation of acceleration attenuation relationships by Esteva [19,20,21], Donovan [10,11], McGuire [31], and Donovan and Bornstein [14,16]. Trifunac and Brady [41] have published relationships but location of a distinct measure of parameter uncertainty in their papers is not possible. Their values are summarized in Table 2. As the relationships are expressed in exponential terms and the variability is known to be lognormally distributed the factor listed is more significant then the standard deviation. This factor is the quantity by which the mean value must be multiplied to find the value one standard deviation higher than the mean value.

TABLE 2

Estimates of Uncertainty

	Author		Standard Deviation Lognormal	Factor
a) acceleration				
	Esteva	1970	1.02	2.8
	Esteva & Villaverde	1973	0.64	1.9
	Donovan*	1973	0.48	1.6
	Donovan	1973	0.71	1.3→1.6
	Donovan & Bornstein**	1975	0.3 →0.5	1.4→1.7
	Seed et al***	1976	0.3 →0.5	1.7
	McGuire	1974	0.51	
b) velocity				
	Esteva	1970	0.84	2.3
	Esteva & Villaverde	1975	0.74	2.1
	McGuire	1974	0.63	1.9
c) displacement				
	McGuire	1974	0.76	2.1

* San Fernando data only
** Site specific relationship
***Seed et al data are sorted by site characteristics and consider only one magnitude level

The values show that when a complete data set is considered without the classification of site conditions the multiplication factor which is used to obtain a value one standard deviation greater than the mean value may be as large as 2. There is not much data published regarding the standard deviation for site specific acceleration data but Donovan and Bornstein estimated that for accelerations on rock and stiff soil sites the standard deviation may be reduced to below 1.5. What this implies is that even when we know what the site conditions are and we know the location of the probable source we still have only a 70 percent chance of measuring a value within plus or minus 50 percent of our computed quantity.

VELOCITY AND DISPLACEMENT

Velocity and displacement have been examined in two different ways. Probably the more common method in use at the present time is a comparison of the peak velocity and displacement with the peak acceleration [24,25,32,39]. Some efforts have been made by Esteva and McGuire to develop direct attenuation equations for peak velocities and displacements. Velocity and displacement data exhibit much more scatter than acceleration and are greatly affected by site conditions but in a different way. Whereas high rock accelerations may be attenuated by a soil profile and small accelerations may be amplified, velocity and displacement values tend to be amplified at most motion levels. The standard deviation factors for some velocity and displacement equations are also included in Table 2.

The direct comparison of peak velocity and peak acceleration was undertaken by Newmark and Hall [33] and has been repeated by others. Site conditions have not been considered by Newmark and Hall, but Mohraz [32] has extended work he initially performed with Hall and Newmark to include these effects. Some of the basic relationships are given in Table 3. While Table 3 shows the similarity of results by different investigators, Mohraz is the only one to show the standard deviation multiplier. These values in Table 3 show that the uncertainties in the quantity ratios from specific events are approximately equal to the uncertainty between the individual quantities themselves. The attenuation relationships proposed by Esteva and McGuire are magnitude dependent and will give different v/a ratios for different magnitude and distance values. The values in Table 3 do not include magnitude and distance, so a comparison of the range of values predicted by use of the mean attenuation equations of Esteva and McGuire could be of value. If the range of magnitudes is varied between 4.5 and 7.0 and epicentral distances are varied between 1 and 65 kilometers (0.6 to 40 miles), then the variation of the v/a ratio for Esteva (1973) is between 81 and 119 cm/sec/g (32 to 47 inches/sec/g) with a mean value of 102 cm/sec/g (40 inches/sec/g). Similar values for McGuire vary between 60 and 133 cm/sec/g (23 to 52 inches/sec/g) with a mean value of 89 cm/sec/g (35 inches/sec/g). The variation in the ratios computed directly from the equations are in accord with seismological observation that the velocity-acceleration ratio should increase with both increasing magnitude and increasing distance from the source.

TABLE 3

Ground Motion Parameter Ratios

Profile Type	v/a Newmark-Hall cm/sec/g	v/a Seed et al cm/sec/g	v/a Mohraz cm/sec/g L*	v/a Mohraz cm/sec/g S*	Standard Deviation Factor Mohraz L	Standard Deviation Factor Mohraz S
Rock	61(24)**	66(26)	61(24)	69(27)	1.58	1.63
Stiff Soil	--	114(45)	--	--	--	--
Deep Sand	--	140(55)	76(30)	91(36)	1.53	1.61
Alluvium	122(48)	--	122(48)	145(57)	1.44	1.49

* Mohraz considered horizontal data in two sets. L comprises the set containing the largest horizontal component from each site and the S set contains the lower value.

** Numbers in parentheses are in units of inches/sec/g.

RESPONSE SPECTRA

Average response spectra including some estimates of the uncertainty have been published by Newmark et al, Blume et al, Seed et al and Mohraz. The procedures for averaging require normalizing some quantity within the response spectra. All investigators have used normalization of the peak acceleration for some portion of their study. Seed et al used this normalization throughout, even in areas of the spectrum where velocity and displacement control. The other investigators used different normalizations for different frequency ranges and then were able to form an average spectrum based on the separate components. The procedures by which the spectra are constructed are reasonably familiar and will not be described here.

The estimates of uncertainty appear to vary slightly across the spectrum but are not great enough to warrant special attention. In Table 4 the average ratio between the mean response spectra coefficients and the mean plus one standard deviation coefficients are shown for three different damping levels. As these spectra are computed from selected data sets and are not based on the total data the standard deviation should be expected to be slightly reduced.

TABLE 4

Spectral Uncertainty

	Averaged Across Spectra		
Damping Level	Blume-Newmark	Mohraz	Seed et al
Average	1.42	1.41	---
	1.37	1.36	1.4
	1.30	1.31	---

SUMMARY

It can be seen that no matter how ground motion input for design is defined a large amount of uncertainty exists. This uncertainty must be recognized and it must be included in the selection of the total criteria used for design. It is imperative, however, that the uncertainty of the whole project be included in the design study rather than compounding the uncertainty of each individual part. Unbridled conservatism could quickly lead to ridiculous criteria. If a conservative acceleration is chosen and then spectra are constructed using conservative ground motion ratios and spectral amplification factors, it is an easy step to end up with criteria that are up to 4.5 times the most probable or mean value. This only relates to the ground portion of design. When each step is considered in this way the use of maximum conservatism is unconscionable . It is important to urge that the uncertainty in design be considered so that the total degree of conservatism is not much greater than that existing in the selection of individual parts. For example, the choice of mean acceleration values and ground motion ratios with conservative spectral amplification values is a useful starting concept.

NEW DIRECTIONS

As the previous section has shown there still exist major unresolved problems relating to the uncertainty of ground motion parameters. Part of the problem has been due to the separation of parts of the problem between disciplines. The seismologist and geophysicist may consider the uncertainty of the representative peak parameters; the earthquake engineer may then ignore the uncertainty in these parameters but consider the uncertainty in the spectral parameters on the basis that the prior numbers are known.

Although it was never explicitly stated it was the intent of Newmark and Hall in recommending the use of mean plus one standard deviation response spectra that mean values would be used for the basic parameters. Fortunately this original intent has been ignored. Page [35] has made the spurious assertion that the largest peak acceleration measured is primarily a function of the number of instruments deployed. It is reasonable to expect to get a larger maximum as the data set increases in size. At the same time as the largest maximum values increase the mean value will become more firmly established. Statistical evaluations and studies must be based on mean values even if they are from a set of maximum values.

There is evidence that some of the apparent scatter of data may be reduced if data sets are processed differently. The ratio between the motion peak and the root mean square value over a carefully defined duration [9,11,28,41] has been used in some probabilistic applications [10,22,30]. Recent studies using r.m.s. techniques for both time histories and response spectra are summarized in a paper prepared for this workshop by Shah and Mortgat [40]. These procedures appear to offer the most promise in better defining typical ground motion parameters for design use by giving more stable quantities.

CONCLUSIONS

It is readily apparent from observation of the values in Tables 2, 3 and 4 that there is a larger degree of uncertainty in presently used techniques of selecting design ground motions. Large projects require the participation of professionals from different disciplines. It is expected that each would apply some measure of conservatism to his recommendations. The cumulative effects of this continued conservatism are rarely considered for the total project. The effects of such accumulations in the geotechnical field have been aptly demonstrated by Peck [36].

Much of the strong dissension that has occurred in the fields of ground motion evaluation and selection has probably come about from research and studies which have examined portions of the seismic problem in great detail while ignoring other possible effects of equally large significance.

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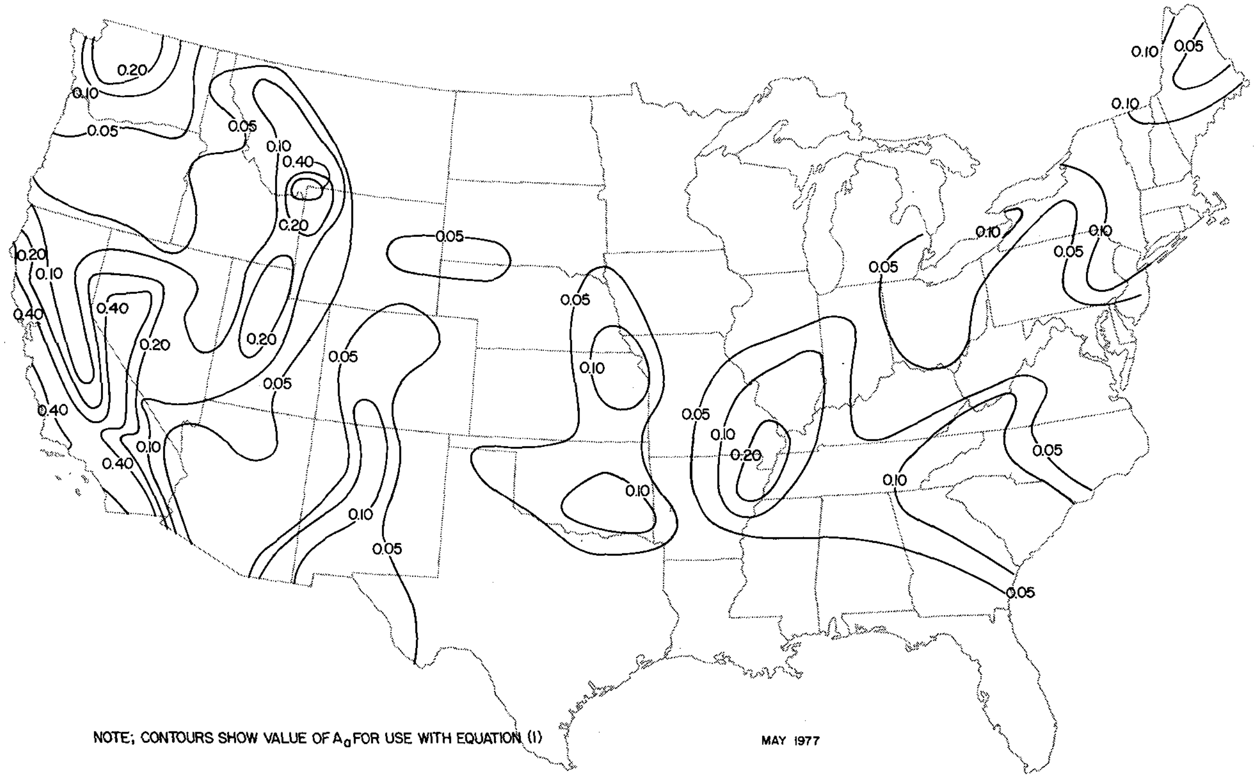

Figure 1: Contour map of Effective Peak Acceleration (EPA) for the 48 contiguous states. Contours represent EPA levels with a non-exceedance probability of between 80 and 95 percent during a 50 year period.

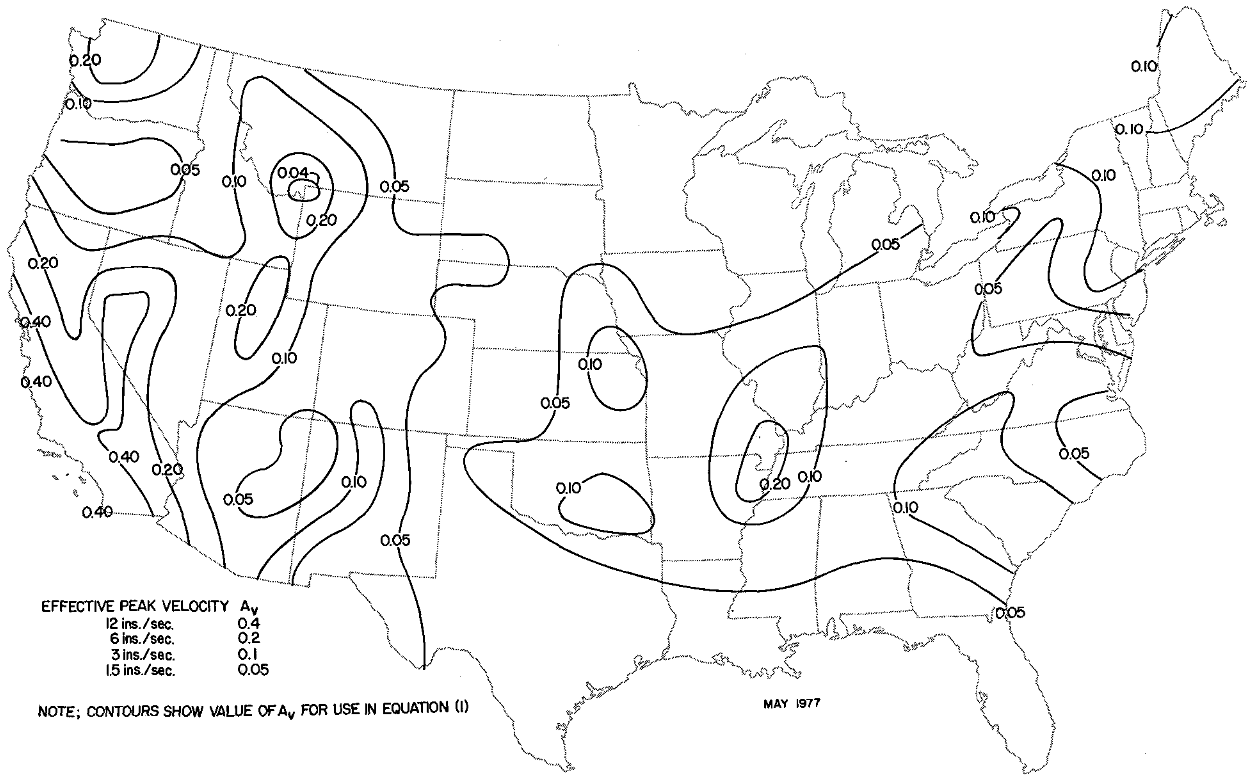

Figure 2: Contour map of Effective Peak Velocity-Related Acceleration (EPV) *for the* 48 *contiguous states.* Contours represent values based on velocity but equivalent to acceleration for use in developing lateral design forces.

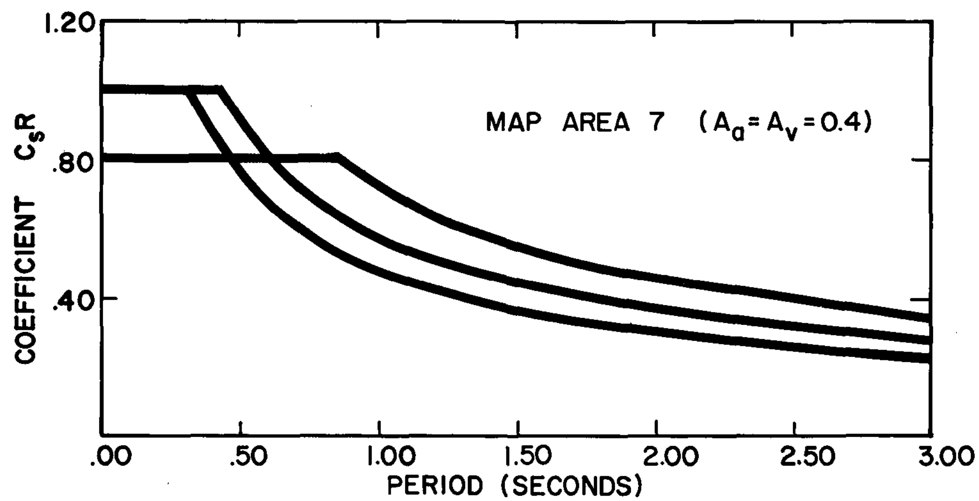

Figure 3: Lateral force coefficient curves for three different soil types.

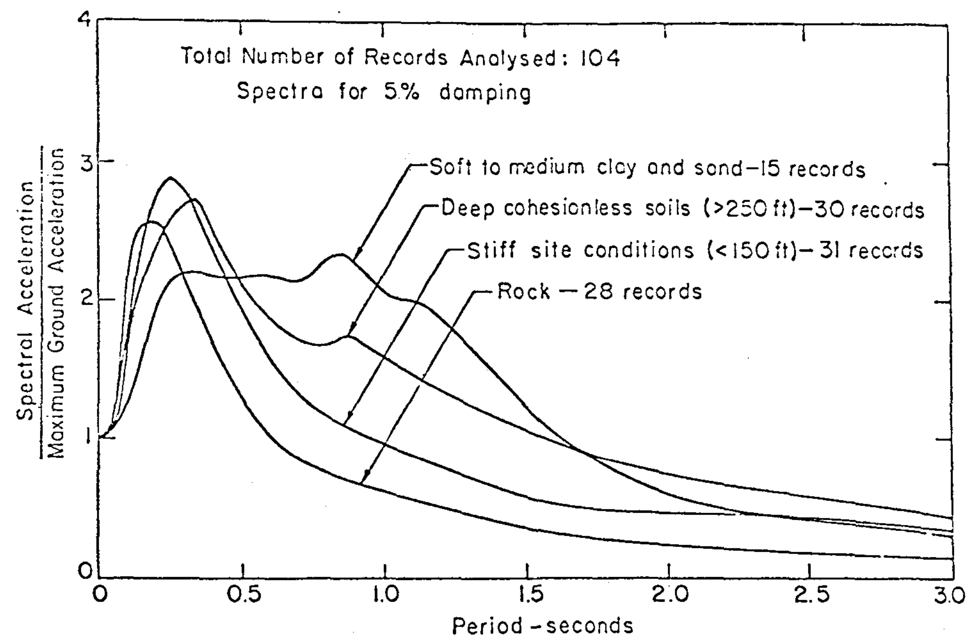

Figure 4: Average Acceleration Spectra for Different Site Conditions.

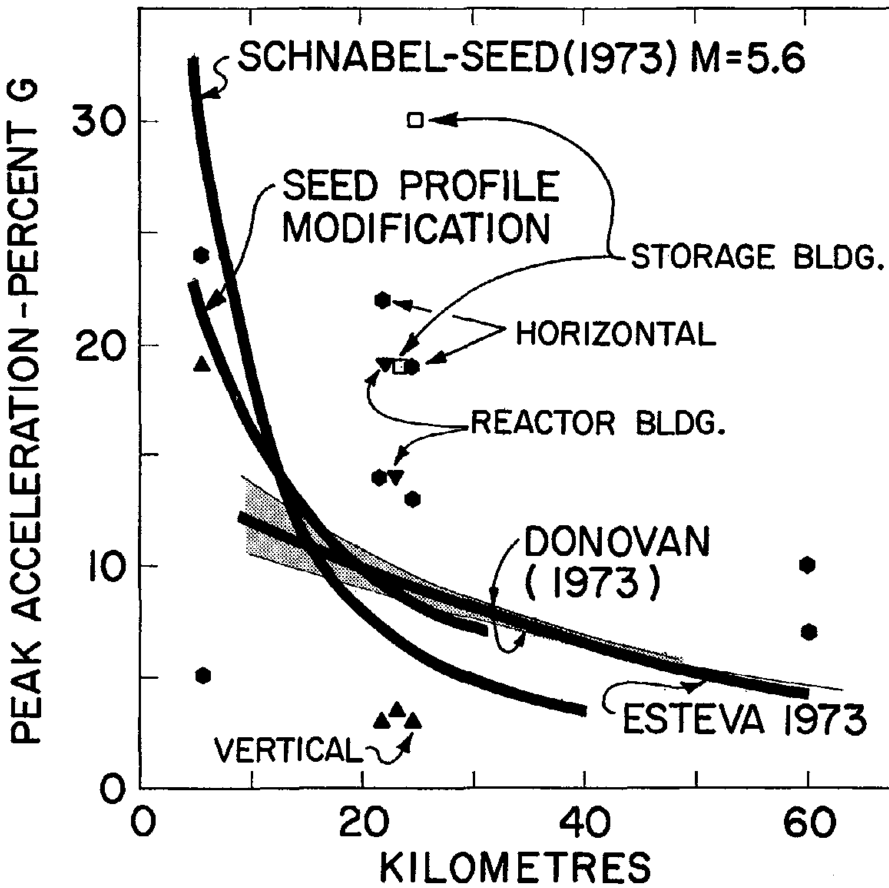

Figure 5: Instrumental Peak Acceleration values recorded during the June 7, 1975 Ferndale Earthquake (Magnitude approximately 5.5 with a 20 kilometer focal depth).

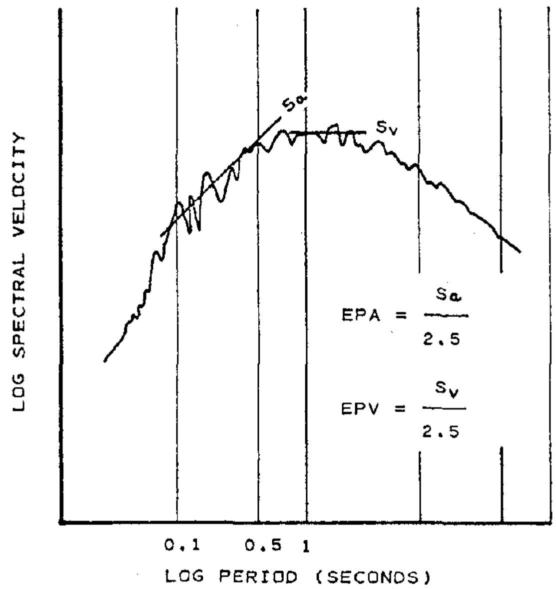

Figure 6: Development of definitions of Effective Peak Acceleration and Effective Peak Velocity from response spectrum values.

STATE-OF-THE-ART IN ESTABLISHING DESIGN EARTHQUAKES

by

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INTRODUCTION

The main objective of this paper is to review the state-of-the-art in establishing design earthquakes by looking at the role of earthquake ground motions in the overall problem of seismic-resistant design of buildings rather than just the isolated problem of predicting ground motions from a geophysical, seismological, soil mechanics or any other specialized point of view.

The need for looking at the establishment of design earthquakes from this overall point of view has been pointed out in a recent paper by Biggs, et al. [1]. In this paper the authors made a partial assessment of the state-of-the-art of seismic design, pointing out that although the past two or three decades have witnessed marked improvements in the analysis of mathematical models subject to seismic input, there has not, unfortunately, been a corresponding improvement in the ability to design structures for earthquakes. Analysis has far outstripped practical utilization of the results for design purposes. The paper indicated that the difficulties encountered in the application of analytical methods to seismic design arise from uncertainties regarding the details of a ground motion and from the inherent sensitivity of the response of the structural system of interest to the detail of the ground motion input. It was further noted that the inadequacies of present methods of seismic design of building structures are not only derived from these uncertainties but that they reflect inadequate consideration of the earthquake risk and associated costs involved.

The need for more comprehensive assessment of risk and cost has also been discussed by Bertero and Bresler [2]. To achieve an optimum design, an estimate of the economic losses resulting from failure is required. The term "failure" as used herein is synonymous with "inadmissible limit states" and includes all modes of undesirable behavior (from superficial damage to collapse) which may render buildings unfit for use. Therefore a logical approach to the seismic design and construction of a structure is that of comprehensive design [2]. In applying this approach it should be recognized that building damage may result from different effects of an earthquake: (1) ground failures due to fault ruptures or those due to the effects of seismic waves (soil vibrations creating fissures, landslides, lurching, nonuniform compaction and associated differential settlement, and liquefaction); (2) vibrations transmitted from the ground to the structure; (3) seismic sea waves (tsunami) and tsunami-like disturbances and seiches in lakes; and (4) other consequential phenomena such as fires, and floods caused by dam failures and by landslides plugging rivers or increasing the water level of lakes.

The effect which usually concerns the structural engineer, and is presently accounted for by seismic-resistant design provisions of building codes, is the response of a structure to ground shaking. This is the only source of seismic damage that will be considered in this paper, but it should be recognized that in evaluating damageability of a building during its service life that the effect of other main factors on both demand and capacity of the building must not be neglected. These factors include aging, changes in use, occupancy, or socio-economic conditions, structural and nonstructural modification, fire damage and repair, corrosion, etc. [2].

According to the above remarks, to achieve an efficient earthquake-resistant building construction, the designer must pay careful attention to the total seismic design and construction process. The various phases of this process begin with evaluation of the seismic threat and representation of the ground motions (establishment of design earthquakes), continue with the selection of proper structural layout and prediction of the mechanical behavior of the whole soil-building system, down to the detailed proportioning and detailing of the structural component together with their connections and supports, and conclude with the final construction and maintenance of the building during its service life (durability).

The main design aspects that should be considered as well as their interrelationships are summarized in the flow diagram of Fig. 1. According to this diagram the first and perhaps most difficult step is the establishment of the design earthquakes.

In this report the review of the state-of-the-art in establishing design earthquakes is carried out by first evaluating present methods. This is followed by a review of studies that have been performed to evaluate the reliability of inelastic design response spectrum methods. Finally, suggestions for future research in this area are offered.

EVALUATION OF PRESENT METHODS OF PRESCRIBING DESIGN EARTHQUAKES

Conceptually, the design earthquake should be that ground motion which is "critical," i.e. which drives the structure to its critical response. The application of this simple concept in practice meets with serious difficulties, however. This is because first, the ground motion is very complex, and secondly, even for a specific structural system, the critical response will vary according to the different limit states that could control a design. Furthermore, the detail of the design earthquake (or ground motion) will depend on the design problem at hand. More specifically, if the problem is to obtain only the design seismic forces for a preliminary design, the design earthquake could be specified in the form of a smoothed response spectra. On the other hand, if consideration is given only to the final design--the proportioning and detailing of the critical regions of a structure--or to study of the reliability of a selected design, it will be necessary to specify time-history ground motions. In this sense the establishment of adequate design earthquakes is analogous to the establishment of proper material stress-strain diagrams for predicting mechanical behavior of structures.

The ground motion experienced at a site is a complex function of the type and characteristics of the source mechanism, the nature of the intervening geological structure, and the topographical and soil conditions near the site.

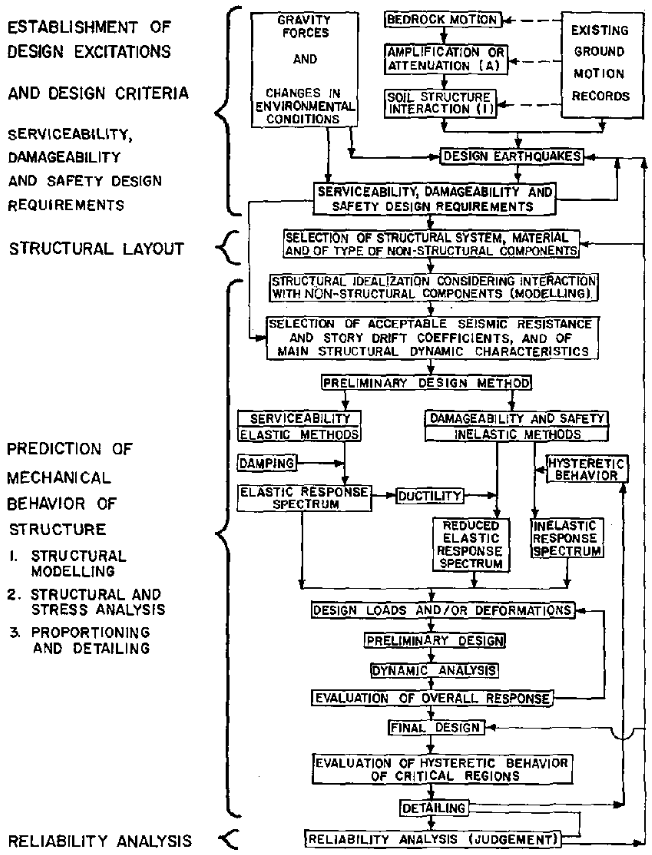


FIG. 1 FLOW DIAGRAM OF GENERAL ASPECTS AND STEPS INVOLVED IN ASEISMIC DESIGN

A common design simplification is to consider only nonconcurrent action of horizontal, translational ground components. For sites near the earthquake source, it may be necessary to base structural response evaluations on the simultaneous action of all six ground components [3] and to consider realistically the nonlinear soil-structure interaction rather than to use predicted free-field ground motions.

Actual records of all ground motion components should be obtained in future earthquakes in order to study their effects on building response and to determine the minimum data required by structural engineers to define design earthquakes.

At present it is very difficult to predict accurately the response of a building to this complex ground motion since, depending on the function and type of structure, different limits of usefulness (limit states) can control the design. In the past it has been recognized that at least two main limit state cases should be considered: one in which the design is controlled by serviceability limit states, and the other, by ultimate limit states. In the former a structure should essentially remain in its linear-elastic range of behavior to avoid functional failure; in the latter, inelastic behavior up to the point of incipient dynamic collapse could be tolerated. Examination of building damage resulting from recent severe seismic ground shaking has revealed that although some buildings were far from reaching the collapse limit state, the degree of nonstructural damage was so great as to constitute failure. It was therefore deemed necessary to consider explicitly in seismic design a third category of limit states based on damageability [2] which would bridge the gap between serviceability at one end and safety against collapse at the other end.

An evaluation of present methods of prescribing earthquakes for each of these main limit states follows.

Serviceability Limit States

Seismic codes have specified design earthquakes in terms of a building code zone, a site intensity factor, or, as in most modern codes, as a peak or effective site acceleration [4]. Reliance on such an acceleration alone, however, is generally inadequate. The following different methods have also been recently suggested: response spectrum, time-history ground motion, and design based on random vibrational analysis. In cases where serviceability limit states control design, structures should remain essentially in their elastic range. For these cases and for structures located at moderate distances from the source, it is generally agreed that one of the best ways to specify the design earthquake is by a smoothed, linear-elastic design response spectrum (LEDRS). Such a spectrum can be constructed from statistical analysis of elastic spectra obtained for appropriate accelerograms (real or simulated), or, by scaling the peak ground acceleration, velocity, and displacement from spectral amplification factors statistically derived for various amounts of damping [5]. When only estimates of peak ground acceleration are available, reasonable values for the peak ground velocity and displacement may be obtained by multiplying the ground acceleration (expressed as a fraction of gravity) by 122 cm/sec. (48.0 in./sec.) and 91 cm (35.8 in.), respectively [6].

As pointed out by Biggs, et al. [1], the only difficulty in using the response spectrum approach lies in the combination of modal components to predict the peak responses. The square root of the sum of the squares (SRSS) of the modal peaks is commonly used with special consideration given to closely spaced modes which may be additive.

The use of a specific time-history seismic ground motion (either actually recorded, normalized to a desired peak intensity, or artificially derived) is attractive because it provides a deterministic result for the selected motion. However, any two motions may produce quite different peak responses even if they have the same intensity and statistical properties. For design purposes this would require analyzing for several motions, resulting in an excessive amount of computations. The use of artificially derived motions, generated from a predicted ground motion response spectrum or a spectral density function, has the advantage over actual recorded motions in that a single record can represent the predicted single-degree response over the entire frequency range. However, this does not eliminate the difficulty in predicting peak responses because two statistically equal, artificial motions remain different in detail and produce different results for multi-degree systems.

The advantage of design based on random vibrational analysis is that it enables the true probabilistic nature of the seismic response to be accounted for. This method easily produces the root mean square (RMS) response in a single mode. However, difficulties arise in predicting the ratio of peak to RMS response and in combining modal response and as such, certain assumptions must be made to obtain the total peak response. Furthermore, the use of a probabilistic model is no less arbitrary than when the model is deterministic [7]; in the former the arbitrariness lies in the assumption concerning the probability distribution underlying the model. Nevertheless, it seems probable that design based on random vibrational analysis will eventually prove to be the most satisfactory approach.

The difficulties encountered in all three methods stem from the same problem, namely, that the details of the time-history ground motion which have an important effect on the response of a multi-degree system, cannot be predicted for a given site.

Biggs, et al. [1], summarized the results of studies at MIT in which a four-degree, shear beam type system was analyzed for a group of 39 actual earthquake accelerograms normalized to 0.3g peak ground accelerations. Comparisons were made among (1) the statistics of the 39 peak responses, (2) the response predicted by the mean response spectrum of the 39 motions, and (3) the response due to 15 artificial motions, all generated from the mean spectrum. Typical results are shown in Table 1. The analysis of these results indicates that:

1. Peak ground acceleration is not a sufficient indication of earthquake effects and use of actual ground motion records is not a reasonable design approach (note the large coefficients of variation, greater than 0.40).

2. Response predicted by a response spectrum analysis using SRSS modal combination based on a mean response spectrum of the 39 accelerograms agrees very closely with the mean of the 39 time-history analyses. The same is true for the mean plus one standard deviation or any other probability level. Thus the much simpler response spectrum method produces a reliable prediction of many motions at the site and eliminates the need for numerous time-history analyses.

3. Means of the 15 response obtained from 15 artificial motions all generated from the mean response spectrum agree very closely with those obtained by the other two methods. Despite identical statistical properties of the artificial motions, however, the coefficients of variation are still large (greater than 0.24). In fact, the ratio of maximum to minimum response for the 15 motions exceeds 2 in all cases. There are two reasons for this variation. First, it is impossible to match exactly the response spectrum, and secondly, for different motions the modes combine differently to produce

TABLE 1
PEAK INTERSTORY ELASTIC DISPLACEMENTS OF FOUR-STORY BUILDING,
FUNDAMENTAL PERIOD = 1.13 SECONDS
(From Reference 1 - Reprinted by permission of Prentice-Hall, Inc., N.J.)

Story	Time-History Analysis of 39 Earthquakes*	SRSS Modal Analysis--Mean (or Mean + σ) Response Spectrum	Time-History Analysis of 39 Artificial Motions†	
1	0.122	0.126	0.133	Mean
2	0.107	0.104	0.155	
3	0.092	0.088	0.093	
4	0.063	0.059	0.064	
1	0.194	0.193	--	Mean + σ
2	0.169	0.166	--	
3	0.137	0.131	--	
4	0.089	0.083	--	
1	0.58	--	0.25	Coefficient of variation
2	0.57	--	0.29	
3	0.48	--	0.29	
4	0.40	--	0.24	

*Normalized to 0.3g peak ground accelerations.
†All generated from mean response spectrum.

the peak response. Thus artificial motions do not solve the problem and if used for design, several must be employed to ensure a safe result.

Response to a particular time-history input may be significantly affected by slight changes in the natural period (T_1) of the structure. Since for real structures T_1 cannot be computed accurately, when time-history ground motions are used for design it is necessary to assume a range of values for T_1 which further complicates the procedure. By using a smoothed response spectrum, the slight changes in T_1 become considerably less important. Because of the large variation that can exist in the estimation of T_1 in actual buildings, however, it is still convenient to use at least the possible bounds of T_1 rather than just one computed value.

In summary, Biggs, et al. [1] concluded that for elastic design, the approach based on the use of a smoothed response spectrum is the most reliable and certainly the most convenient. Design based on random vibrational analysis is of interest because of its rationality, but further development is required before practicing engineers will be comforable with this approach. As pointed out by Donovan [8], the danger of complex design procedures is that they can give a false sense of achievement. Thus for cases where serviceability limit states control design, the most effective way of defining the design earthquake is through the use of an LEDRS.

Simple methods suggested for the construction of such a spectrum have been based on so-called standard severe earthquake motions at moderate distances from the causative fault. For building sites located near such faults, however, the LEDRS should be based on the actual maximum values that can be expected for the parameters defining the ground spectrum: effective ground acceleration, velocity, and displacement [9]. These values should be determined from analysis of available records and/or from theoretical predictions based on the faulting process at the causative fault. Estimates of the peak ground velocity and displacement obtained by multiplying the expected ground acceleration by suggested coefficients obtained from analysis of standard earthquake ground motions alone can lead to unconservative values of LEDRS [9]. If no records are available for sites near causative faults, and if acceptable predictions of the effective peak values for the ground acceleration, velocity, and displacement cannot be made, then establishment of the critical earthquake ground motion should be based on techniques suggested by Drenik, Wang, and Wang [10], or Hoshiya, Shibata, and Nishiwaka [11].

Further studies on the subject of spectral amplification factors for different amounts of damping are needed. Significant differences were found between the values of the ratio of maximum elastic responses corresponding to different amounts of damping and those corresponding to presently suggested amplification factors [9].

Ultimate Limit States

It is generally not economically feasible to design buildings near faults for the forces indicated by LEDRS. Lower design forces may be used if it is possible to take advantage of a building's ability to absorb and dissipate energy by inelastic deformations. To ensure safety against collapse or to avoid large economic losses due to damage, however, inelastic deformations must be kept within acceptable limits.

One of the most urgent needs in ERCBC is the development of a reliable, yet practical, design procedure based on inelastic behavior considering the two main categories of the ultimate state design, that is, damageability and collapse. In developing this procedure, one of the main problems is to establish reliable design earthquakes.

The design of conventional buildings according to code requirements anticipates inelastic behavior during severe earthquakes although the design is normally based on elastic analysis. Current code procedures based on equivalent static force and elastic analysis are not satisfactory.

At present only time-history analyses offer reasonable prediction of the response of multi-degree-of-freedom (MDOF) systems--models--in the inelastic range. Problems in using time-history motions as input for practical preliminary design include the difficulties in reliably modeling hysteretic behavior of a real building and the computational effort required. This effort is considerably greater than required in the case of linear-elastic design because different mechanical models of the expected hysteretic behavior (or at least their bounds) must be considered. Moreover, because the variability in response to different possible ground motions is considerably greater than in the elastic case, design cannot be reliably based on a single motion.

To emphasize the importance of the variability in response to different possible motions, the results obtained at MIT and reported by Biggs, et al., in Reference 1 are briefly discussed. The four-degree, shear beam type system considered in the elastic analyses, whose results are presented in Table 1, has been assumed to have at each story an elasto-plastic resistance function with a yielding resistance at each story proportional to the first mode elastic story shears.

Typical results, in the form of interstory ductility ratios, obtained under the same 39 recorded and 15 artificial time-history ground motions considered for the elastic case of Table 1 are shown in Table 2. Comparison of the results presented in these two tables indicates:

TABLE 2
PEAK INTERSTORY DUCTILITY RATIOS OF FOUR-STORY BUILDING,
FUNDAMENTAL PERIOD = 1.13 SECONDS
(From Reference 1 - Reprinted by permission of Prentice-Hall, Inc., N.J.)

Story	Analysis of 39 Earthquakes*	Analysis of 15 Artificial Motions†	
1	5.7	4.4	Mean
2	2.6	3.2	
3	4.0	5.0	
4	9.7	13.8	
1	1.23	0.42	Coefficient of Variation
2	0.48	0.31	
3	0.48	0.28	
4	0.49	0.39	
1	38.6-0.8	10.2-2.6	Max.-Min.
2	5.9-0.8	5.6-1.9	
3	8.9-1.0	7.6-2.9	
4	27.8-2.2	21.1-7.0	

*Normalized to 0.3g peak ground acceleration.
†All generated from mean response spectrum.

1. The means for the two sets of ground motions differ considerably in each story, although the average over the four stories are similar.

2. The ductility ratios are much higher in the bottom and top stories. This behavior is not predicted by elastic analysis, indicating another inherent difficulty in design for inelastic behavior, namely, that of achieving uniform, or any other designed, ductility ratios throughout the structure.

3. The coefficients of variation are generally larger in the inelastic case than in the elastic case.

4. The differences in the maximum and minimum responses for both sets of motions are rather dramatic. This further illustrates that any two

motions, although presumed similar, may produce radically different inelastic responses and any particular motion may be unconservative for design.

Analysis of the individual responses to the 39 recorded motions shows that the yielding structure has an "effective inelastic period" longer than the elastic period. While little correlation was found between the peak response and the ordinate of the response spectrum at the elastic period, there is some correlation at the effective inelastic period. Designers should be cautious in cases where a lengthening period may result in greater elastic response.

This last observation has led some researchers to suggest the possibility of using elastic response spectrum to predict peak inelastic response by estimating the expected effective inelastic period. Unfortunately, this effective period cannot be easily predicted, particularly in the case of MDOF systems.

To further study the reliability of present code procedures, another MIT study [1] is considered. In this case, several typical buildings were designed according to the UBC and then analyzed to determine the inelastic response due to a strong artificial ground motion. The time-history of this motion was generated from postulated elastic response spectrum. Five designs were made for each building corresponding to UBC zones 0, 1, 2, and 3 plus a zone 4 with a seismic coefficient of 2. The results of Fig. 2 indicate that with one exception, CSW-11 (a shear wall building with relatively short natural period), an increase in the design zone had little effect in reducing the required amount of average interstory ductility. The results also showed even less reduction in the peak interstory displacements and a very poor distribution of yielding over the height of the building. Thus it was concluded that:

1. Increasing the design zone does little to reduce damage in a strong earthquake. The writer would like to add that this observation applies only to the strong ground motion considered in these analyses. As will be discussed later, the use of other severe earthquakes with different detailed dynamic characteristics (severe, long duration pulses) may have shown the opposite effect.

2. The code procedure does not provide the designer with effective means of improving the building's performance since s/he is not given direct control over the response parameters (peak interstory displacement and ductility ratios) causing damage.

An improvement over the simple code procedure is to specify the design earthquakes for ultimate limit states through the use of inelastic design response spectra (IDRS).

Preliminary design loads can be obtained from IDRS derived by evaluating the nonlinear dynamic response of structural models with realistic hysteretic idealizations subjected to various ground motions with characteristics appropriate to the site, e.g. see Reference 12. Simpler methods which directly modify LERS to obtain IDRS using factors based on the elasto-perfectly plastic response of single degree-of-freedom (SDOF) systems [6] are more

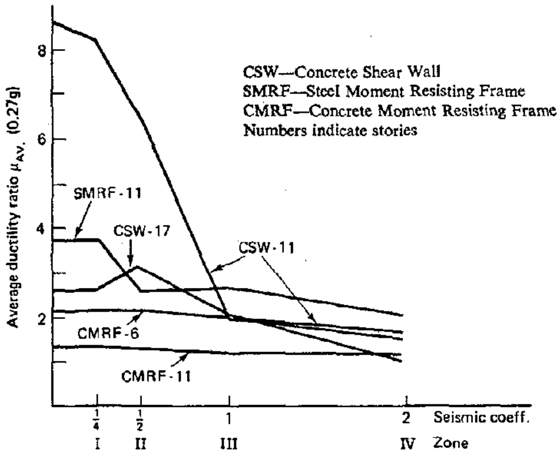


FIG. 2 EFFECT OF DESIGN ZONE ON AVERAGE DUCTILITY RATIO (From Ref. 1 - Reprinted by permission of Prentice-Hall, Inc.,N.J.)

commonly used [13,14]. The use of these types of IDRS permits to design for specified ductility and drift ratios. However, these methods are based on limited numbers of ground motion records, and, as their proposers have pointed out, they should be used with caution when applying them to sites that can be subjected to significantly different kinds of ground motions. Furthermore, such methods may not be suitable for MDOF systems, or in cases where the actual hysteretic behavior is likely to differ from the assumed elasto-plastic idealization [6,9]. A valuable discussion on the basis and limitations of these and other more precise methods for constructing IDRS directly from LEDRS can be found in References 15-18.

The IDRS derived from response spectra of SDOF systems does not eliminate the difficulty of achieving uniform (or any other desired pattern) yielding and story drift over building height. The seriousness of this problem was demonstrated by results of studies carried out at MIT and Berkeley. In the MIT studies summarized in Reference 1, simple shear beam models, designed by IDRS derived as suggested by Newmark and Hall [6] were analyzed to obtain the inelastic response to an artificial motion matching the design spectrum. A typical result is shown in Fig. 3. While the average interstory distortions are very close to the design values, the distribution over the building height is far from uniform. No satisfactory means for controlling this distribution was found. The problem is further complicated by the sensitivity of the results to the assumed resistance function, which cannot be predicted with confidence. Repeating the analyses using a trilinear function, the distribution was slightly improved, but when a stiffness-degrading model was employed, excessive distortions were computed in both the top and bottom stories.

A new design procedure proposed by the Applied Technology Council (ATC) and experimentally applied to several buildings in the ATC-2 project [13] utilizes IDRS derived from LERS and also attempts to control local member ductility ratios and interstory drifts. For practicability, however, the method is based on elastic modal analysis, and ductility ratios are computed on the basis of the peak elastic distortion and the yield limit distortion. As pointed out by Biggs, et al. [1], this procedure is questionable since local, inelastic distortions may be quite different.

The studies carried out at Berkeley [9,19-22] show that the validity of

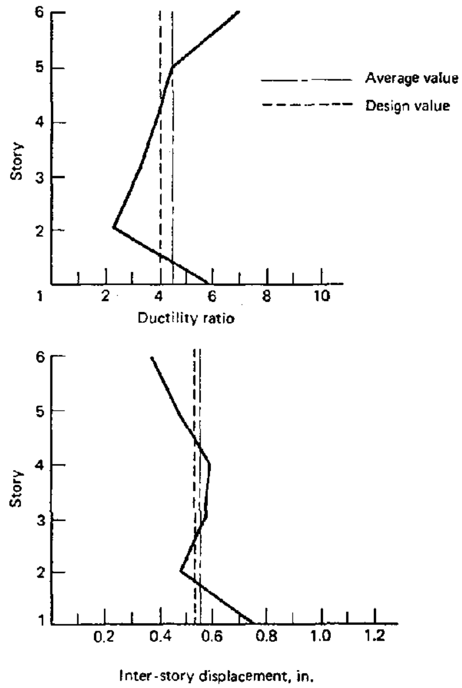


FIG. 3 PEAK INTERSTORY DISPLACEMENTS AND DUCTILITY RATIOS FOR BUILDING DESIGNED BY IDRS (From Ref. 1 - Reprinted by permission of Prentice-Hall, Inc., N.J.)

deriving IDRS directly from the LEDRS can be seriously questioned because the types of excitations that induce the maximum response in elastic and inelastic systems are fundamentally different. The information used for computing, and therefore contained in, LEDRS, although necessary, is insufficient for predicting the maximum inelastic dynamic response. This information should be complemented with data on the duration of strong ground shaking and the number, sequence, and characteristics of intense, relatively long acceleration pulses (i.e. pulses resulting in large ground velocity increments) that can be expected.

Duration of strong ground shaking--Before discussing the reasons for having this information, it is necessary to define more specifically what constitutes strong ground shaking and how the duration of this intense part of shaking can be established. No unique level of ground acceleration can be established as the threshold of strong ground shaking because this level depends on many factors. Of these, the most significant are the dynamic characteristics of the ground motions and of the building, and the yielding strength of the building. The problem is complex because both elastic and, particularly, inelastic responses are sensitive to the interrelation of these characteristics. The inelastic response is sensitive to the possible deterioration in the dynamic characteristics and yielding strength of the building with the history of its hysteretic behavior. In determining the duration of the strong motion, the possibility of having one or more aftershocks should be considered.

It has been argued that the only information necessary for computing LEDRS is the estimation of the peak response (in this case, maximum displacement ductility), which is not very sensitive to the duration of ground motion. Although results obtained in a study at Berkeley [12] using four artificial accelerograms with different periods of duration show that the influence of these different periods was not large, quantitative results obtained in other studies recently conducted at Berkeley [9, 19-22] and MIT [23] have clearly shown the opposite. A review of the basic principles governing hysteretic behavior and failures of actual structures under generalized dynamic excitations (such as those expected from earthquake ground motions) also show the important role that the duration of strong motion can have.

Failures under generalized dynamic excitations--Collapse of a structure can occur as a consequence of "low-cycle fatigue" or "incremental deformations" under excitation intensities lower than those required to induce instantaneous collapse if these excitations are considered as monotonically increasing [24]. As pointed out in References 2 and 25, cumulative damage resulting from a long, strong ground motion, a short main shock followed by a sucession of aftershocks, or a combination of the main shock and another consequential event or environmental exposure such as fire, can lead to either one of the above two phenomena and therefore merits considerably more attention than it has received.

Yamada and Kawamura [26] have discussed an ultimate aseismic design philosophy of reinforced concrete based on low-cycle fatigue. This type of failure is very sensitive to detailing and quality control of materials and workmanship used in construction. If errors in design or construction, or lack of quality control of materials and of workmanship are eliminated, then application of adequate seismic design provisions with possible further improvements [27], will result in structural designs in which low-cycle fatigue would not control the design. By detailing the expected critical regions of different structural members according to recently proposed seismic code provisions for preventing sudden tensile failure of the steel, delaying the inelastic buckling and preventing early failures due to shear or to crushing of confined concrete, the energy absorption and energy dissipation capacity developed under cyclic reversals of deformation having maximum intensity will be so large as to resist the energy input of even the toughest of credible seismic motions. Even under the most severe ground motions recorded, the number of reversals that can occur between opposite peak deformations having the maximum intensity is not usually large enough to be of serious concern [27]. It should also be noted that under full reversals of symmetrically yielding and strain-hardening or strain-softening structures, the P-Δ effect is cancelled out (Fig. 4).

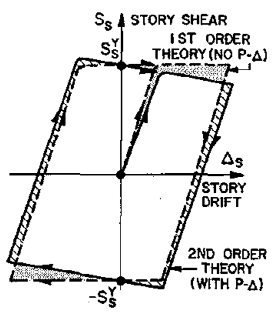


FIG. 4 EFFECT OF P-Δ ON HYSTERETIC BEHAVIOR INVOLVING FULL DEFORMATION REVERSALS [2]

Studies carried out at Berkeley have shown that one case where low-cycle fatigue could control the design involves members that are used as structural dampers to dissipate energy. One typical example of such a case is that involving coupling girders in coupled wall systems [28]. However, failure of these members does not necessarily lead to complete structural failure. Since these elements act as safety fuses between two different structural resistant systems, their failure would lead to a change in the dynamic characteristics of the system rather than to a brittle failure of the complete system. Low-cycle fatigue can be a serious problem in structures that rely only on energy absorption and dissipation throughout shear deformation mechanisms and/or in bond slippage mechanisms.

A schematic illustration of the incremental collapse, denoted as "crawling collapse," is shown

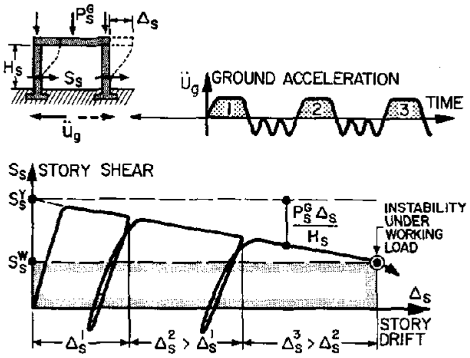


FIG. 5 P-Δ EFFECTS ON INCREMENTAL COLLAPSE TYPE OF RESPONSE INDUCED BY A SERIES OF SEVERE ACCELERATION PULSES [2]

in Fig. 5. Recent studies [9] have shown that this type of failure can control the aseismic design of structures, particularly those at sites near the source of seismic ground motions containing severe, long acceleration pulses. For example, the study of the response of a multistory steel frame, optimally designed using a nonlinear method, to seismic ground motions derived from those recorded during the 1971 San Fernando earthquake shows that the frame will collapse due to the type of incremental deformations illustrated by the first story displacement time-history response of Fig. 6. The danger of incremental collapse is aggravated by the high probability that several aftershocks of intensities and dynamic characteristics comparable to the main shock will occur. As Newmark and Rosenblueth [25] have pointed out, it is not unusual for a structure which is able to withstand a major shock with visible damage, to collapse during an aftershock.

Although the P-Δ effect is not a factor in failures due to low-cycle fatigue, it is of paramount importance in failures of an incremental collapse type. As a structure is deflected away from its original vertical equilibrium position, the increment in sidesway deflection under repetition of the same acceleration pulse will increase since the structure's available net yielding resistance against lateral inertial forces is considerably reduced by the P-Δ effect (Fig. 5). Accumulation of these increasing incremental deflections can lead to an instability phenomenon under a working load combination (gravity forces plus wind or minor earthqukes). Assuming that increasing deformations and numbers of reversal cycles may lead to deterioration in the actual strength of the structure, the instability problem can be considerably aggravated in actual buildings.

Number, sequence, and characteristics of intense, long-duration acceleration pulses--The need for this information is evident in the results obtained by applying the vibration theory to SDOF systems [19]. In the linear-elastic case, the critical dynamic excitation is of a periodic type having a frequency equal to that of the system which induces an engineering resonance phenomenon. For this type of excitation, the dynamic magnification operator, D, can reach a maximum value approximately equal to $1/2\xi$. Thus, for values of ξ ranging from 2% to 10%, D can attain values ranging from 25 to 5. Since the largest value of D for an impulsive excitation is only 2, severe long acceleration pulses are not usually critical for linear-elastic response.

In an inelastic system, such long pulses can become critical. This is particularly true for a structure having a hysteretic yielding resistance, R_y, equal to or less than the inertial force corresponding to the effective ground acceleration of the pulse, $\ddot{X}_e$, i.e. $R_y \leq M\ddot{X}_e$, where M is the mass of

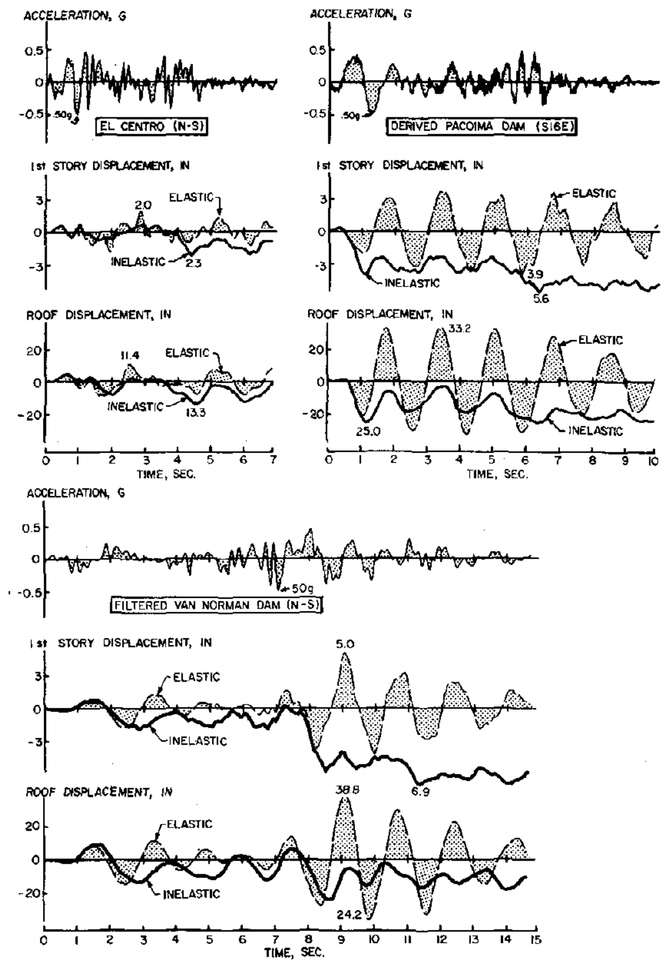


FIG. 6 RESPONSE OF 10-STORY FRAME TO DIFFERENT ACCELEROGRAMS [9]

the structure. In the case of elasto-plastic systems, the existence of periodic, short acceleration pulses in the ground motion contributes only to building the response of the system up to its yielding level. Once the system begins to yield, the phenomenon of engineering resonance is depressed since the energy dissipation through even small inelastic deformations is equivalent to very large values of ξ. Therefore, large inelastic deformations are not expected during each yielding excursion. Although the existence of periodic short pulses can induce a series of yielding reversals, it is doubtful that the number of these reversals can lead to the phenomenon of low-cycle fatigue. This is because the amount of inelastic strain developed in each reversal will usually be so small that the number of reversal cycles required to induce fracture would exceed the number which can occur, even in the longest conceivable strong motion of an actual earthquake. This is so assuming that the mechanism of energy dissipation is of a flexural type and that inelastic buckling of the main reinforcement is restrained.

The above discussion indicates that the amplification factors to be applied to the maximum ground accelerations in order to obtain the maximum linear-elastic response of a structure are usually controlled by the engineering resonance phenomenon. On the other hand, considerably larger inelastic deformations can be induced by the presence of just one long pulse with an effective acceleration equal to or just greater than that corresponding to the yielding strength of the structure. Furthermore, repeated applications of severe, long acceleration pulses can lead to the accumulation of sufficiently large inelastic strains, which could induce one or a combination of the two types of failure discussed above, i.e. low-cycle fatigue or incremental (crawling) collapse. Of the two, the author believes the latter to be the critical failure against which the structure should be designed.

It should be clear from the above discussion that the design earthquake is not unique, even for a given site. As already pointed out, the critical ground motion depends on the type of behavior that is expected to control the response of the building at the site or on the limit states controlling the design.

From results already available on the response of SDOF systems to impulsive forces, it is clear that in the case of seismic ground motions the larger the intensity of the effective acceleration of a pulse with respect to the yielding strength of the structure, the shorter the rise time to the peak acceleration and the longer the duration of the pulse relative to the fundamental period of the structure, the larger the amount of inelastic deformations that will develop. However, in order to specify quantitatively the inelastic design earthquake, it is necessary to determine (1) the severity of the long acceleration pulses that can be developed during an earthquake, and (2) the manner in which these pulses can be defined. An attempt to resolve these problems follows by analyzing the few existing records in which these kinds of pulses have been observed.

Analysis of 1971 San Fernando Earthquake Records

Severity of long acceleration pulses--It is possible to address this problem by analyzing the records of the two strongest motions obtained from the San Fernando earthquake February 9, 1971. The only strong motion accelerograph record near the fault rupture of this earthquake was obtained at Pacoima Dam (PD). A seismoscope record was also obtained at the abutment of the lower Van Norman Dam (VND), which was located near the fault zone.

Pacoima Dam record--This record (Fig. 7a) contains the highest ground acceleration registered to date, 1.25g. Several investigators [29,30] have indicated that the irregular surface topography in the vicinity of the

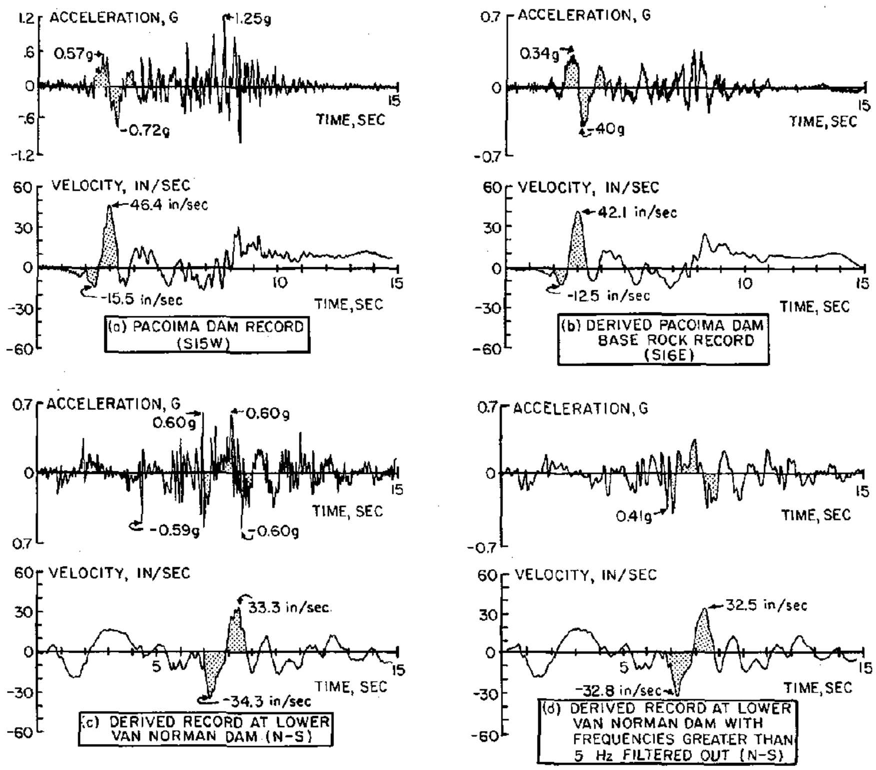


FIG. 7 SAN FERNANDO EARTHQUAKE NEAR-FAULT GROUND MOTION RECORDS [22]

accelerometer significantly affected the frequency content of the record, especially for frequencies greater than 1 Hz. A series of analyses of the dam and its adjacent geological structure led to a derivation of the ground motion at sites below the base of the dam (Fig. 7b) [30]. Since the objective of the analysis was to remove the effects of local surface topography and interaction of the dam with its foundation from the original record, the derived record is probably more representative of ground motions at other nearby sites than the actual PD accelerogram. It should be noted, however, that the derived record was based on an erroneous orientation initially reported for the PD record, i.e., the S-15°-W component was originally identified as S-16°-E [29].

Examination of the derived Pacoima Dam (DPD) record (Fig. 7b) indicates that the high peak accelerations registered in the PD record after 6 sec. may not be characteristic of ground motions experienced at other nearby sites. Both the actual and derived records, however, exhibit three severe acceleration pulses, each of about 2/3 sec. duration, between 2 and 4 sec. These unusual acceleration pulses resulted in very large ground velocities (Fig. 7) and incremental ground velocities [PD, 1.57 m/sec. (61.9 in./sec.); DPD, 1.39 m/sec. (54.6 in./sec.)]. They were also responsible for the unusually large linear-elastic response spectrum values for periods longer than 0.8 sec. (Fig. 8).

Van Norman Dam records--The ground motion necessary to produce the seismoscope trace obtained at the abutment of the lower VND [located near the fault zone, about 10 km (6 mi.) from PD] has been estimated [31]. The north component of this record is shown in Fig. 7c. Although many of the characteristics of this ground motion differ from those of the PD records, as would be expected, the ground motion exhibited a series of severe, long acceleration pulses that led to large ground velocity increments [1.72 m/sec. (67.6 in./sec.)]. These long-duration acceleration pulses become more evident when frequencies above 5 Hz are filtered from the VND record, shown in Fig. 7d.

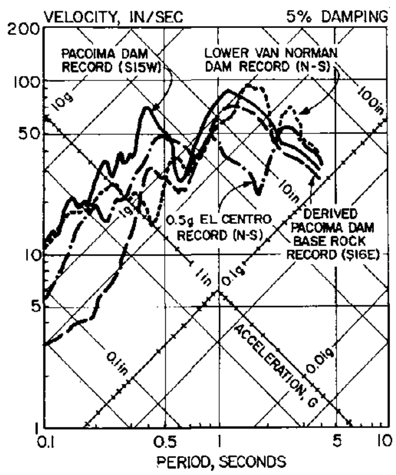


FIG. 8 LINEAR-ELASTIC RESPONSE SPECTRA [9]

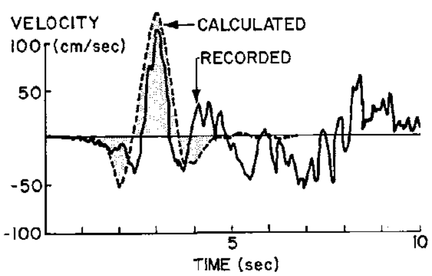


FIG. 9 COMPARISON OF THEORETICAL GROUND VELOCITY FOR STICK-SLIP FAULTING WITH PD RECORD [3]

Characteristics of near-fault records--It may be possible to determine the characteristics of long-duration acceleration pulses by examining records of near-fault ground motions. Similar ground motion characteristics have been reported for several other earthquakes at sites on firm ground close to the fault zone [25]. Analytical studies based on simple two- and three-dimensional fault dislocation models [29,32] have verified that the near-fault ground motions of the San Fernando earthquake were characterized by large ground velocity pulses of the type exhibited by the records in Fig. 8. These pulses are directly related to the faulting process and are not a result of local geological conditions. Studies of stick-slip faulting [33-35] have also indicated that such wave forms are not unique to thrust faulting (Fig. 9). Such studies have led Boore and Zoback [30] to conclude that peak particle velocity may be a better basis for establishing design earthquakes than peak acceleration, and that the initial portions of the PD records containing the large velocity pulse may be appropriate for seismic-resistant design of structures located close to potential faults.

Little empirical data are currently available relating the peak ground acceleration and velocity for epicentral distances less than 15 km (9.3 mi). Theoretical limits [36,37] of the peak near-fault particle velocity have been placed in the range of 1.0 - 1.5 m/sec. (40-59 in./sec.). Newmark and Hall [6] have also indicated that it is unlikely for the maximum ground velocity to exceed 1.2 - 1.5 m/sec. (48-60 in./sec). No estimates have yet been made for the maximum incremental velocities or the associated peak accelerations. Another important factor to be determined is the minimum acceptable rise time for each of these severe, long-duration accelerations.

Romanian Earthquake of 4 March 1977 [38]

A trace of a copy of a record obtained from a strong motion instrument (SMAC-B 1967) installed at the Romanian Building Research in Pantelimon, in the northeast sector of Bucharest, is shown in Fig. 10. The site where this

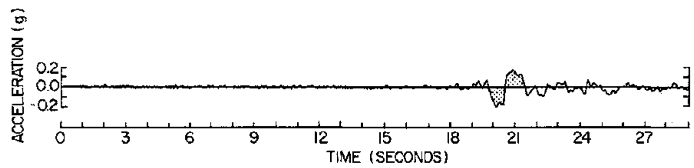


FIG. 10 STRONG-MOTION ACCELEROGRAM RECORDED IN BUCHAREST, N-S COMPONENT [38]

record was obtained was located about 165 km (103.1 mi) from the epicenter with a 200 km (125 mi) slant distance from the focus. The focus (hypocenter) was estimated at a depth of 110 km (68.8 mi), and the earthquake had magnitudes of $m_b = 6.8$ and $M_s = 7.2$. Analysis of the trace of Fig. 10 indicated several interesting features of the ground motion in the N-S direction: (1) an unexpected pulse-like motion (acceleration); (2) the severe pulse occurred about 20 sec. after the instrument was triggered (triggering level is 0.01g vertical acceleration), which, according to Bolt [38], suggests that the pulse was the S wave from the source; (3) the pulse, of a shape resembling a sine wave, had a peak acceleration of 0.20g and a period of about 1.7 sec., i.e., each half-wave had a duration of about 0.85 sec.

According to this preliminary data the unexpected pulses had an incremental ground velocity on the order of 120 cm/sec. (47 in./sec.) which is very high for a site located at a distance about 200 km (125 mi) from the focus. If the reliability of the above Bucharest record can be established, it will be of great seismological and engineering importance because it will offer proof that severe, relatively long-duration acceleration pulses can also occur at great distances from faults. Thus the establishment of design earthquakes in the form of smoothed response spectra (elastic and inelastic) based on response ground spectra derived from the dynamic characteristics of "standard" earthquake records (such as El Centro, Taft, etc.) would be questionable, not only for structures located near faults, but also for those located at large distances from the fault.

STUDIES CARRIED OUT TO EVALUATE THE RELIABILITY OF IDRS DERIVED FROM PROPOSED LEDRS

Analytical Studies of Olive View Earthquake Damage

Evidence of the effects of severe, long-duration acceleration pulses contained in actually recorded and analytically derived earthquake motions was obtained from the analytical studies of the damage induced in the newly constructed buildings of the Olive View Medical Center. These buildings suffered extensive damage during the San Fernando earthquake, despite seismic resistance coefficients far in excess of then existing code requirements [39]. For example, the six-story main building had story seismic resistance coefficients exceeding 0.3; the permanent drifts [greater than 0.76 m (30 in.)] and the associated damage suffered by this reinforced concrete building were so large that it had to be demolished.

An extensive field, laboratory, and analytical investigation has been conducted to identify the factors that controlled the behavior of the main building [40]. Although some of the local damage to the buildings was found to be the result of the inadequate structural system, poor member detailing and deficient construction workmanship, analyses of the building indicated that the overall damage pattern and the large residual displacements observed were primarily a consequence of severe, long-duration acceleration pulses like those experienced at the Pacoima and Van Norman Dams.

The analytical results obtained in the study presented in Ref. 40 indicate that the response of yielding structures is very sensitive to severe, long-duration acceleration pulses such as those present in the near-fault records of the San Fernando earthquake. Thus the following additional studies were conducted.

Nonlinear Dynamic Analysis of Single and Multi-Degree-of-Freedom-Systems

Several nonlinear dynamic analyses of (SDOF and MDOF systems) were performed to assess the reliability of present methods of constructing IDRS from LEDRS for near-fault sites in view of possible severe, long acceleration pulses. These results are compared with those for the N-S component of the 1940 El Centro earthquake, which is often considered representative of standard strong motion records.

SDOF systems--The basic equilibrium equation controlling the motion of a viscously damped, SDOF system subjected to a ground acceleration time-history, $\ddot{u}_g$, is given by:

$$M\ddot{u} + 2M\omega\xi\dot{u} + R = -M\ddot{u}_g \qquad (1)$$

in which M is the mass of the system; ξ is its viscous damping ratio; ω is the system's natural circular frequency; R is the force resisted by the system; and $\ddot{u}$ and $\dot{u}$ are the system acceleration and velocity, respectively, at any time.

To obtain useful design charts for nonlinear structures, it is desirable to rewrite eq. 1 in a nondimensional form which accounts for yielding. By noting that $K = \omega^2 M$ and $R_y = Ku_y$, by introducing variable transformations $\mu = u/u_y$ and $\rho = R/R_y$, and by expressing the ground acceleration as a fraction of the peak ground acceleration in the record, $\ddot{u}_{g_{max}}$, eq. 1 can be written as:

$$\ddot{\mu} + 2\xi\omega\dot{\mu} + \omega^2\rho = -\frac{\omega^2}{\eta}\frac{\ddot{u}_g}{\ddot{u}_{g_{max}}} \tag{2}$$

In the above equation, the value of η is the ratio of the seismic resistance coefficient to the peak ground acceleration expressed as a fraction of gravity:

$$\eta = \frac{R_y}{M\ddot{u}_{g_{max}}} = \frac{C_y}{\ddot{u}_{g_{max}}/g} \tag{3}$$

in which g is the acceleration of gravity, and C_y is the system's seismic resistance coefficient, i.e. the yield resistance, R_y, divided by weight of the system, M•g. The nondimensional hysteretic response of a nonlinear system (μ and ρ) to a particular nondimensionalized ground motion $[\ddot{u}_g/\ddot{u}_{g_{max}}]$, can thus be evaluated in terms of η and the parameters ω and ξ needed for an elastic system. From this evaluation, it is possible to construct charts in which the required displacement ductility, μ, of an SDOF system to a given ground motion can be plotted as a function of ξ, T and η.

Several elasto-perfectly plastic SDOF systems, with 5% damping and with periods ranging from 0.1 to 2.0 sec., were subjected to the DPD, the original and filtered VND, and the El Centro records. For each period, the response was computed for various values of the parameter η. Semilogarithmic plots of the absolute value of the maximum displacements divided by the system's yield displacement (displacement ductility factors) are shown in Fig. 11.

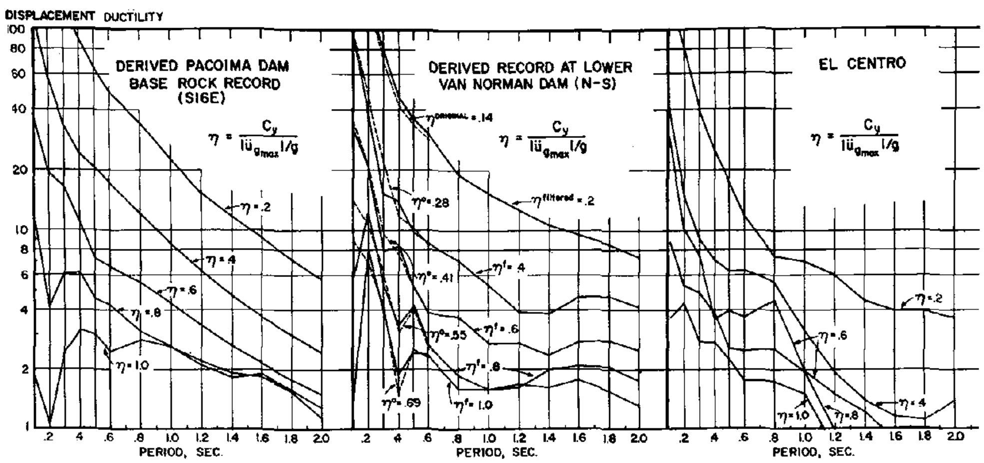
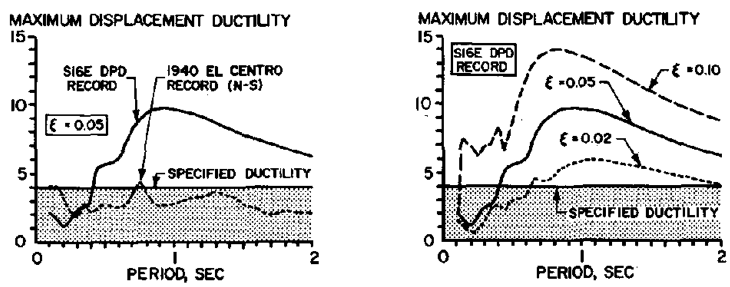


FIG. 11 DISPLACEMENT DUCTILITY REQUIREMENTS FOR SDOF SYSTEMS WITH 5% DAMPING [9]

using such plots for a given ground motion record, μ can be determined if T, ξ, C_y, and $\ddot{u}_{g_{max}}$ are known. Alternatively, the value of C_y required to obtain a desired value of μ can be calculated if T and ξ of the system and $\ddot{u}_{g_{max}}$ of the given ground motion are known. For the ground motions considered, ductility demands generally increased with decreasing values of η and period. For any given value of η, the ductility demands for both the DPD and VND records were generally much greater than for the El Centro record, except when η approached unity in the short period range.

It is evident from Fig. 11 that if the ductility demands are to be kept at acceptable levels, η must be nearly unity in the short period range for any of these ground motions and that it must be maintained close to this value at much longer periods for the DPD and VND records than for the El Centro record. Required ductilities increase rapidly as η becomes smaller than unity, especially for the DPD and VND records. For the level of C_y currently required by building codes, very large ductility factors will result if ground motions like those considered in Fig. 11 with peak ground accelerations greater than 0.3g occur, especially for short period buildings. Furthermore, if IDRS based on effective ground accelerations smaller than the expected effective peak values or on ground velocities obtained assuming standard ground spectrum shapes [13] are used for structures near active faults, undesirably large ductilities could result. For example, the actual ductility requirements for elasto-perfectly plastic SDOF systems with 5% viscous damping designed according to the IDRS in Reference 6 for a desired $\mu = 4$ are shown in Fig. 12a for the El Centro and DPD records. While the displacement ductilities required for the El Centro record are generally smaller than those predicted by the IDRS, ductilities required by the DPD record exceeded the specified value by factors as great as 2.2 for periods longer than 0.4 sec. IDRS based on $\mu > 4$ are even less reliable for near-fault motions.

Derivation of IDRS directly from LEDRS erroneously assumes that increasing damping is as beneficial to the response of inelastic systems as it is to elastic systems. It has been found that the spectral amplification factors used to construct LEDRS [6] may significantly overestimate the effect of damping on inelastic response [9], resulting in lower design forces than actually required to achieve a given μ. The typical effect of this is illustrated by Fig. 12b which shows that ductility requirements for elasto-perfectly plastic SDOF systems designed using suggested IDRS [6] for a $\mu = 4$ increase with increasing values of the viscous damping ratio, ξ.



(a) for different records (b) for different damping ratios

FIG. 12 COMPARISON OF SPECIFIED AND REQUIRED DUCTILITIES [21]

MDOF systems--A three-bay, ten-story frame was designed according to a five-step computer-aided procedure [41], which tries to achieve an economical and practical minimum weight design that is serviceable, and safe from collapse during a severe earthquake. Design forces for the safety design were obtained from an IDRS for a peak acceleration of 0.5g, a displacement ductility factor of 4 and a damping ratio of 5%.

The designed frame had a C_y value of 0.18, and a first mode period of 1.67 sec. At this period, the pseudo-velocity used in its design was 0.38 m/sec. (14.8 in./sec.), 31% higher than the value of 0.29 m/sec. (11.4 in./sec.) corresponding to current IDRS recommendations [6].

Elastic and inelastic models of the frame were subjected to the normalized El Centro, DPD and filtered VND accelerograms with 0.5g peak accelerations. The results of the roof and first floor displacement obtained for these models, as well as the corresponding input accelerograms, may be seen in Fig. 6. Inelastic response to the DPD and filtered VND motions resulted in displacements considerably larger than those to El Centro; as much as 1.9 times larger at the roof, and 2.4 times larger at the first floor. Permanent deformations in the frame were substantially larger.

Although the elastic and inelastic responses to El Centro were generally similar, no such similarity was observed for the DPD and filtered VND records. The dissimilarity in the responses is more striking for the DPD record, where the large acceleration pulses occurred early in the input accelerogram.

The response of this frame shows that elastic response cannot be reliably used to predict peak inelastic response. The elastic results overestimated the peak inelastic roof displacements by more than 25% and underestimated the peak inelastic first story displacement by more than 40% for the DPD and VND records. Furthermore, the type of inelastic response history expected from ground motions with long acceleration pulses is characterized by a few large displacement excursions rather than numerous, intense oscillations as observed in the elastic analyses. Thus interpretation of possible inelastic behavior from elastic response analysis alone could lead to an erroneous conclusion that low-cycle fatigue could be a problem for this frame.

With respect to the results obtained for SDOF systems, it should be noted that for a system with a period of 1.67 sec., the C_y value of 0.18 (closer to the upper bound for all 3 records), the displacement and ductility requirements were unacceptably large. Such differences between the SDOF systems and the example frame are to be expected since analyses of SDOF systems neglect the effects of gravity loads, geometric nonlinearities, etc. Furthermore, the lateral load-deflection relationship for multistory frames is not generally elastic, perfectly plastic. Extrapolation of results from SDOF to MDOF systems should be done with great caution [18].

Very similar results confirming the above conclusions have been obtained in the analysis of a reinforced concrete, ten-story, three-bay frame, whose optimum design is discussed in a paper by Zagajeski and Bertero [42].

RESEARCH NEEDS IN ESTABLISHING DESIGN EARTHQUAKES

In concept the design earthquake should be that ground motion which drives the structure to its critical response. It has been shown, however, that application of this simple concept in practice met with serious difficulties. Because even for a given site and a specific structural system the design critical response will vary according to the different limit states controlling design, at least the following states should be considered: serviceability, damageability, and collapse. The main observations derived from the discussion presented herein and research needs for establishing the design earthquakes for each of these states follow.

General Observations

The following are applicable for design earthquakes based on any of the governing limit states.

1. Strong-motion instrumentation capable of recording all 6 components, particularly at near-fault sites, should be installed. Only the continued accumulation of statistical evidence can lead to improved estimates of the severity of ground motions at the foundation of a building.

2. The effect of each of the 6 components, acting independently, as well as simultaneously, on the elastic and inelastic response of buildings with different structural systems must be analyzed. At present little guidance is available regarding specification of the simultaneous input motions. This is true even in the simplest case where only the two horizontal, translational components are considered.

Design Earthquakes for Serviceability Limit States

1. Linear-elastic design response spectra offer relatively simple and reliable methods for specifying design earthquakes governed by serviceability requirements.

2. The use of "standard" LEDRS should be done with care. At near-fault sites, ground spectrum shapes based on strong-motion records obtained at moderate source distances may significantly underestimate the peak ground velocity and displacements. Realistic spectral shapes based on analyses of available near-fault records, or from theoretical predictions accounting for the faulting process and the nonlinear mechanical characteristics of a building's foundation media, should be used. The record obtained in Bucharest during the Romanian earthquake indicates that even at sites located some distance from the earthquake source, the use of ground response spectra based on values obtained from analysis of only standard ground motions can lead to unconservative LEDRS.

3. The effect of equivalent linear viscous damping, ξ, on spectral amplification factors requires further study, particularly in cases involving pulse-like ground motions.

Design Earthquakes for Ultimate States: Damageability and Collapse

1. One of the most pressing problems in establishing design earthquakes

for ultimate states concerns whether damage or collapse of nonstructural and/or structural elements is used as the criterion for acceptable deformations and, in each case, to determine the type of deformation inducing the event. This is a problem directly related to the assessment of earthquake risk and associated costs and requires further study.

2. Present code procedures are inadequate for specifying design earthquakes based on ultimate states.

3. Use of IDRS permits the designer to have control over the response parameters which cause damage and collapse; however, the use of IDRS derived from recommended LEDRS through displacement ductility factors suggested by present methods, appears to be unconservative for buildings located in the immediate area of causative faults or in general at sites where ground motions containing severe, long acceleration pulses can occur.

4. The effect of equivalent linear viscous damping, ξ, on the response of elastic and inelastic systems should be thoroughly investigated. It has been found that the spectral amplification factors used to construct LEDRS may significantly overestimate the effect of ξ on inelastic response to pulse-like ground motions. Even in the case of elastic response the presently suggested amplification factors overestimate the effect of ξ.

5. Inelastic response cannot be inferred directly from elastic response, since the ground motion characteristics which govern maximum elastic and inelastic responses are generally different. Thus, methods that obtain IDRS directly by modifying LEDRS may not be reliable.

6. Derivation of rational and reliable IDRS requires full characterization of the expected severe ground motions at the site. This requires estimation of the duration of strong ground shaking and the number, sequence, and characteristics of intense, relatively long acceleration pulses. The LEDRS and the derived IDRS do not account for the duration of strong motions during major earthquakes. Information in this area is needed to determine the maximum inelastic deformation excursion, as well as the maximum number of reversals of inelastic deformations, for the structure's critical regions. Such data are essential for the proportioning and detailing of these regions. Although information has recently become available on the duration of strong shaking for certain areas [43], data for most seismic regions of the U. S. remain scarce.

7. Unusually large ground velocities may be developed, particularly at near-fault sites. Methods for constructing IDRS (as well as LEDRS) should reflect the larger values recorded at such sites.

8. Research is needed to establish bounds on the different parameters that define the dynamic characteristics of severe long pulses, i.e. the largest incremental velocity and the associated effective acceleration that can be developed according to the dynamic mechanical characteristics of the soil present at a site. These values will enable the design engineer to determine an upper bound on the energy that can be transmitted to the foundation of the structure so that the structure can be designed accordingly. The

need for improving knowledge of the dynamic soil characteristics cannot be overemphasized. Unfortunately, the trend among soil mechanics researchers appears to be toward application of analytical tools to predict dynamic response of soils, rather than investigation of soil properties.

9. For any given building site it is necessary to know the number of long, severe pulses that can occur during an earthquake since repeated pulses can lead to incremental (crawling) collapse of the building.

10. Because of the inherent sensitivity of the response of a structural system to the details of the ground motion input, analysis of the reliability of any design should be performed using several time-history ground motions whose dynamic characteristics cover all possible motions that could be expected at the building site.

11. At present it is common to evaluate the reliability of a seismic-resistant design by analyzing the designed structure under one or more ground motions obtained by normalizing recorded earthquake accelerograms to some maximum selected value of the peak acceleration. Unfortunately, these ground motions are often the result of earthquakes with different magnitudes, recorded at sites located at different distances from the earthquake source, and having different soil conditions. Indiscriminate use of such a technique, when significant inelastic behavior is expected under severe ground motions, can lead to highly misleading results. For example, accelerograms obtained on soft soil at sites distant from the earthquake source usually contain very long pulses. If these accelerograms are normalized to a large peak acceleration, these pulses may become unrealistically severe, as shown in Fig. 13 where the E-W component of the accelerogram recorded at the ground level of the Orion Avenue Holiday Inn during the San Fernando earthquake is normalized to 0.5g. The record was obtained at about 21 km (13 mi.) south of the epicenter of the earthquake, and the geological source data indicate that the site lay on recent alluvium [44].

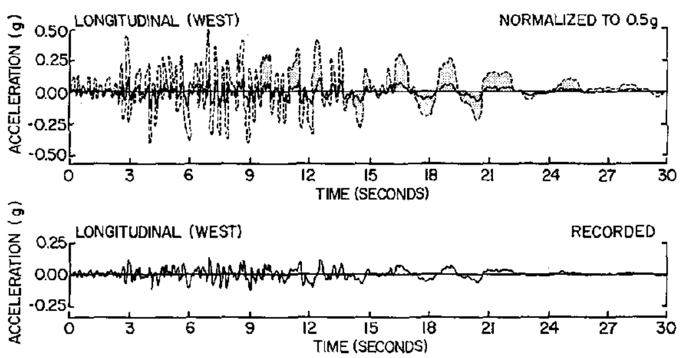


FIG. 13 LONGITUDINAL (E-W) COMPONENT OF GROUND MOTION AT ORION AVENUE HOLIDAY INN, NORMALIZED TO 0.5g [44]

12. The presence of severe, relatively long acceleration pulses, particularly in near-field records, substantially increases the spectral velocity and, more importantly, the required seismic resistance coefficient, C_y, of buildings, particularly those with relatively long periods. To illustrate this, the values of C_y needed to limit ductility to 4 for the El Centro, DPD, and VND records (normalized to 0.5g peak accelerations) are compared in Fig. 14 with current IDRS and code values. Unless the values of μ and ξ usually assumed in design can be substantially increased, structures located at near-fault sites must be designed for much higher forces than currently specified in codes. Although IDRS require sufficiently high C_y values in the short period range, these values are underestimated for near-fault records at periods greater than 0.5 sec.

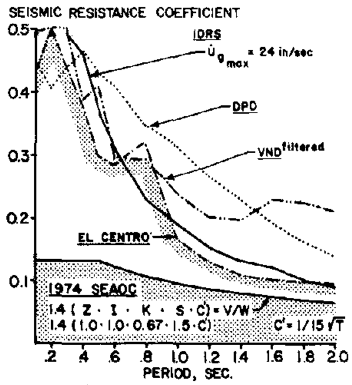


FIG. 14 SEISMIC RESISTANCE COEFFICIENT, C_y, FOR 0.5g PEAK ACCELERATION GROUND MOTION, ξ = 5% AND μ = 4 [9]

13. Although structures can be detailed to accommodate the large ductilities that might result at near-fault sites if they are designed using current codes or IDRS forces, this may be undesirable except for short period structures. The danger of underestimating design forces at near-fault sites was illustrated by the performance of the main building of the Olive View Hospital during the 1971 San Fernando earthquake.

14. Obtaining all the information considered necessary for the establishment of reliable design earthquakes under ultimate states will entail extensive investigation and research. Until this is done, the following procedure may be implemented:

For the case of SDOF systems, charts similar to those presented in Fig. 11 should be prepared. These charts should consider the different hysteretic models (at least the bounds of possible stiffness degradation and strain-hardening) and all earthquake ground motions previously recorded at sites near faults as well as those which can be obtained from theoretical consideration of fault mechanisms. Once sufficient records are available, statistical analysis of the results obtained should be conducted in order to formulate inelastic design earthquakes in the form of IDRS (C_y vs. T, as illustrated in Fig. 14). This will require the establishment of acceptable ductility factors.

Since ultimate limit state design criteria are not only controlled by the energy dissipation capacity of the structural system, but also by damageability, i.e. by the deformations that can be tolerated due to economic, safety, or stability considerations, selection of displacement ductility based solely on the former may be insufficient to establish design earthquakes. Current methods usually recommend the use of a constant ductility. The selection of a design ductility factor without considering structural period or earthquake type (magnitude, source distance, duration, etc.) is unacceptable. Even for a specific structural system, however, the amount of acceptable ductility will vary depending on whether nonstructural or structural damage controls the design. If design is controlled by nonstructural damage, the allowable ductility will decrease with increases in the flexibility (period) of the selected structure. Since present methods do not distinguish between the types of damage controlling a design, the first step in formulating inelastic design earthquakes as an IDRS should be to seek more reliable methods for establishing values of acceptable ductility.

Comprehensive studies to determine more rational methods for establishing acceptable ductilities, particularly for flexible structures, are needed. Investigations are also needed regarding the economic impact of designing structures for either seismic resistance coefficients or design ductility ratios higher than those presently assumed.

In searching for more rational values of ductilities or seismic resistance coefficients, it is necessary to examine the uncertainties involved in selecting the values of all the parameters pertinent to the design process. To do this, the interrelationship of these parameters must be considered; isolated studies of each parameter are insufficient. For example, in designing for strength both sides of the basic design equation should be considered. On one side, there are the computed internal forces as determined from the critical design excitations; on the other, there is the strength of the structural elements. Present methods of designing sections, regions, and whole structural members include the use of several factors which usually lead to significant overstrength [42]. Thus, by taking a conservative approach, looking at each side of the design strength equation independently, one may arrive at an unreasonable and uneconomical overconservatism.

The charts derived for SDOF systems may be used only as design guidelines in the case of MDOF systems. The response of different MDOF systems to severe ground motions such as those resulting from the San Fernando earthquake should be extensively investigated to determine ways in which IDRS obtained for SDOF systems can be modified for MDOF systems, or to formulate new procedures for establishing design earthquakes for the inelastic design of the latter.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION
University of California, Berkeley, July 11-15, 1977

UNCERTAINTIES IN SEISMIC INPUT AND RESPONSE PARAMETERS -- DEVELOPMENT OF STABLE DESIGN PARAMETERS

by

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I. INTRODUCTION

In a recent survey by the EERI Committee on Research Needs, the membership of this prestigious organization felt that the research need in the general area of seismic risk was of utmost importance. It is a well-known fact that the level of seismic safety that a community (or a society) requires depends on the acceptable level of risk for that community and the corresponding costs in achieving that level of risk. Thus, before a rational and acceptable building regulation or code can be developed, a proper understanding of the uncertainties associated with the seismic phenomenon and its consequences should be evaluated. There have been various attempts to "solve" this problem of seismic hazard, seismic risk, and the concept of acceptable risk. The authors of this proposal will not attempt to go into the specifics of various reports and papers written in this general area. A list of such references is given at the end of this paper. However, the general problem and the approach taken by various researchers will be outlined here.

Expected hazard and expected risk have an implication of future uncertainty. Hence, it is not surprising that principles of probabilistic forecasting and decision-making are used by various researchers and risk analysts in evaluating seismic risk for a given region or site.

Consider the Uniform Building Code seismic zone map (Fig. 1). Note that the title of the map indicates "Seismic Risk Map of the United States." This map indicates, to some scale, the future seismic hazard in different parts of the country. It cannot and does not take into account any consequences due to future seismic events. Also, this map does not take into account the frequency of occurrence of earthquakes. In spite of these shortcomings, many engineers perceive or evaluate seismic "risk" through these maps.

Another widely used concept of seismic risk evaluation is that of "maximum credible event" or "maximum probable event." This perception or definition of seismic risk is generally used by geologists and seismologists. This concept has great value for truly unique and important structures such as nuclear reactors and dams. However, for most structures, this concept has certain shortcomings. First, it does not give information regarding the probable level of "loadings" during the economic life of the structure. Second, it has no frequency of occurrence information in it. Third, it is overly conservative to design structures to either "maximum credible" or "maximum probable" events which were based on geological information for geological time spans, though

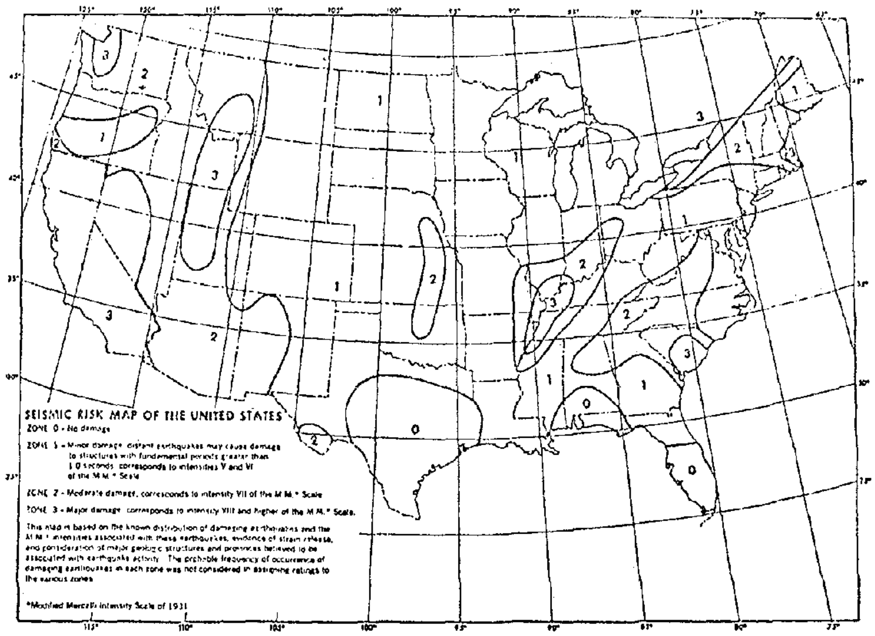

SEISMIC RISK MAP OF THE UNITED STATES
ZONE 0 - No damage
ZONE 1 - Minor damage, distant earthquakes may cause damage to structures with fundamental periods greater than 1.0 seconds; corresponds to intensities V and VI of the M.M.* Scale
ZONE 2 - Moderate damage, corresponds to intensity VII of the M.M.* Scale
ZONE 3 - Major damage, corresponds to intensity VIII and higher of the M.M.* Scale.
This map is based on the known distribution of damaging earthquakes and the M.M.* intensities associated with these earthquakes, evidence of strain release, and consideration of major geologic structures and provinces believed to be associated with earthquake activity. The probable frequency of occurrence of damaging earthquakes in each zone was not considered in assigning ratings to the various zones.
*Modified Mercalli Intensity Scale of 1931
115°
110°
105°
100°

Figure 1

the economic life of the structure might be only 50 to 200 years. Also, the consequences of failure of many structures are not severe enough to warrant a level of design (with unreasonably small risk) at the maximum credible levels. An example of this type of risk map is shown in Fig. 2. This map was developed by Roger Greensfelder of the California Division of Mines and Geology.

In recent years, a considerable amount of work is done in the area of probabilistic estimation of seismic load parameters. In particular, probabilistic forecasting in terms of iso-acceleration or iso-intensity maps has been developed by many researchers. Figs. 3 and 5 are two typical examples. The analytical model in developing such maps is based on the following parameters and assumptions (see Fig. 3).

1. Point, line, or area seismic sources are identified, based on past seismic and geological data.

2. For each potential source, a recurrence relationship is developed. This could be linear, bilinear, or nonlinear. Various such relationships are suggested in the literature.

3. Forecasting of future events is made by using either a Poisson or Markov model, the most commonly used model being Poisson.

4. A suitable attenuation relationship linking the intensity or peak acceleration or any other peak value parameter with magnitude and distance is used to obtain the probable "loading" at a site. This has been a very "weak link" in the overall formulation, because the scatter of the peak parameters is too great to give statistically reliable estimates. More will be said about this aspect in the next section.

The "risk maps" developed in the above fashion have the following informational content in them.

- They include all the seismological data available for all the potential seismic sources.

- Such maps take into account the frequency of occurrence of various levels of seismic events.

- The probabilistic representation of the "loading" parameter gives the designer some idea about the risk he is taking in designing a structure for a specific "load" level.

- The probable loading during the economic life of the facility can be explicitly represented.

In spite of the above advantages, researchers and designers have realized some of the shortcomings of such maps, which are supposed to give a clear perception of seismic risk. The following are some of the shortcomings of these risk maps.

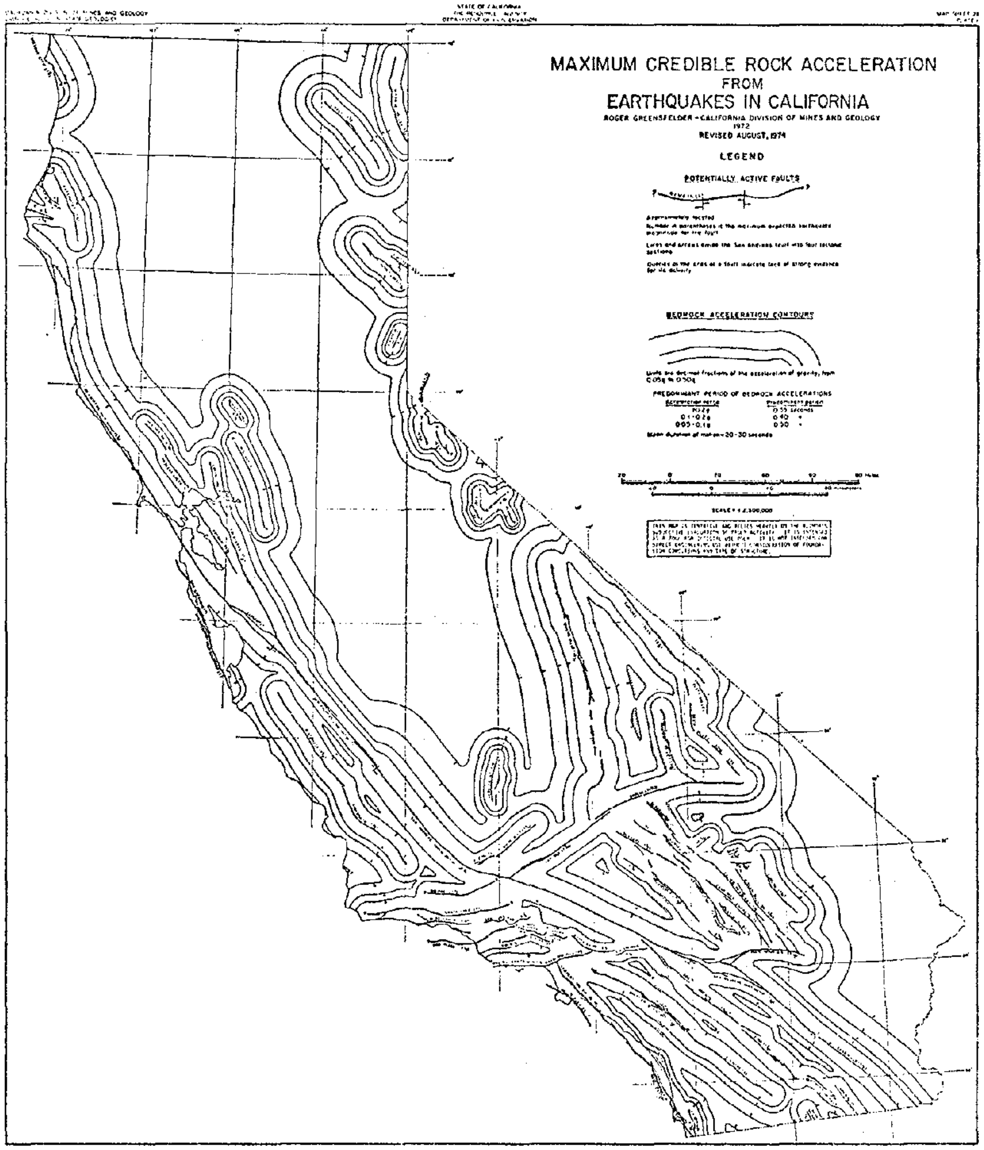

MAXIMUM CREDIBLE ROCK ACCELERATION
FROM
EARTHQUAKES IN CALIFORNIA
ROGER GREENSFELDER - CALIFORNIA DIVISION OF MINES AND GEOLOGY
1972
REVISED AUGUST, 1974
LEGEND
POTENTIALLY ACTIVE FAULTS
BEDROCK ACCELERATION CONTOURS
PREDOMINANT PERIOD OF BEDROCK ACCELERATIONS
Mean duration of motion - 20 - 30 seconds
SCALE 1:2,500,000

Figure 2

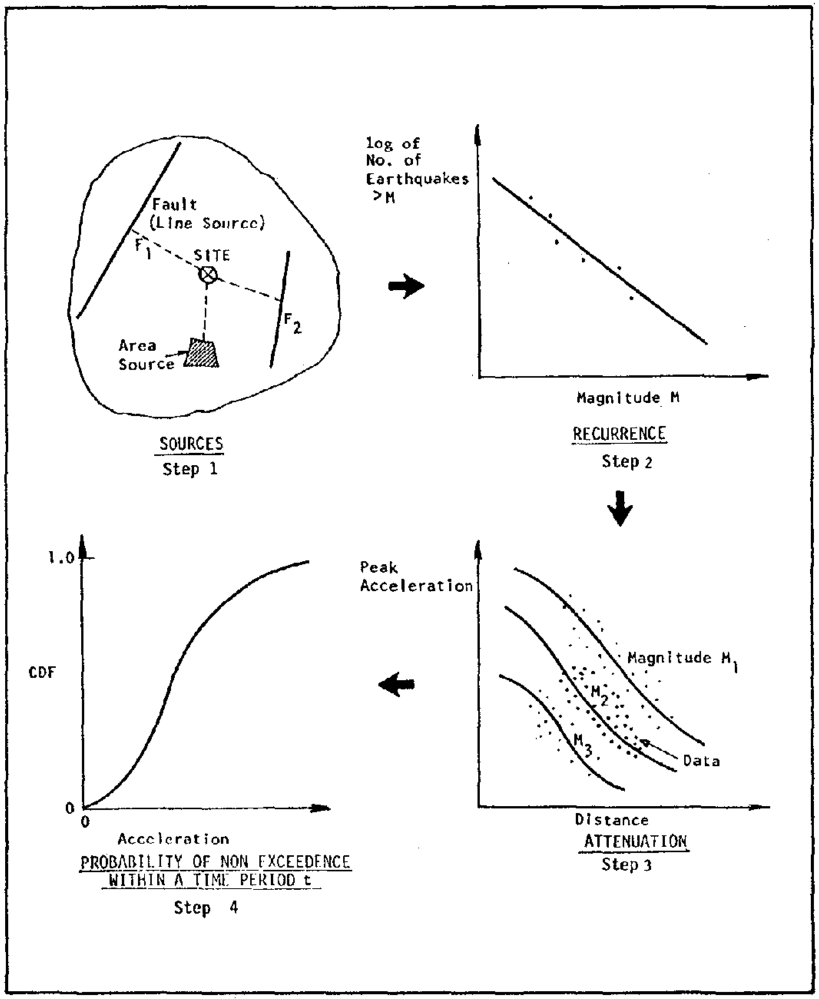


Figure 3. Current Approach to Hazard Mapping for Peak Values

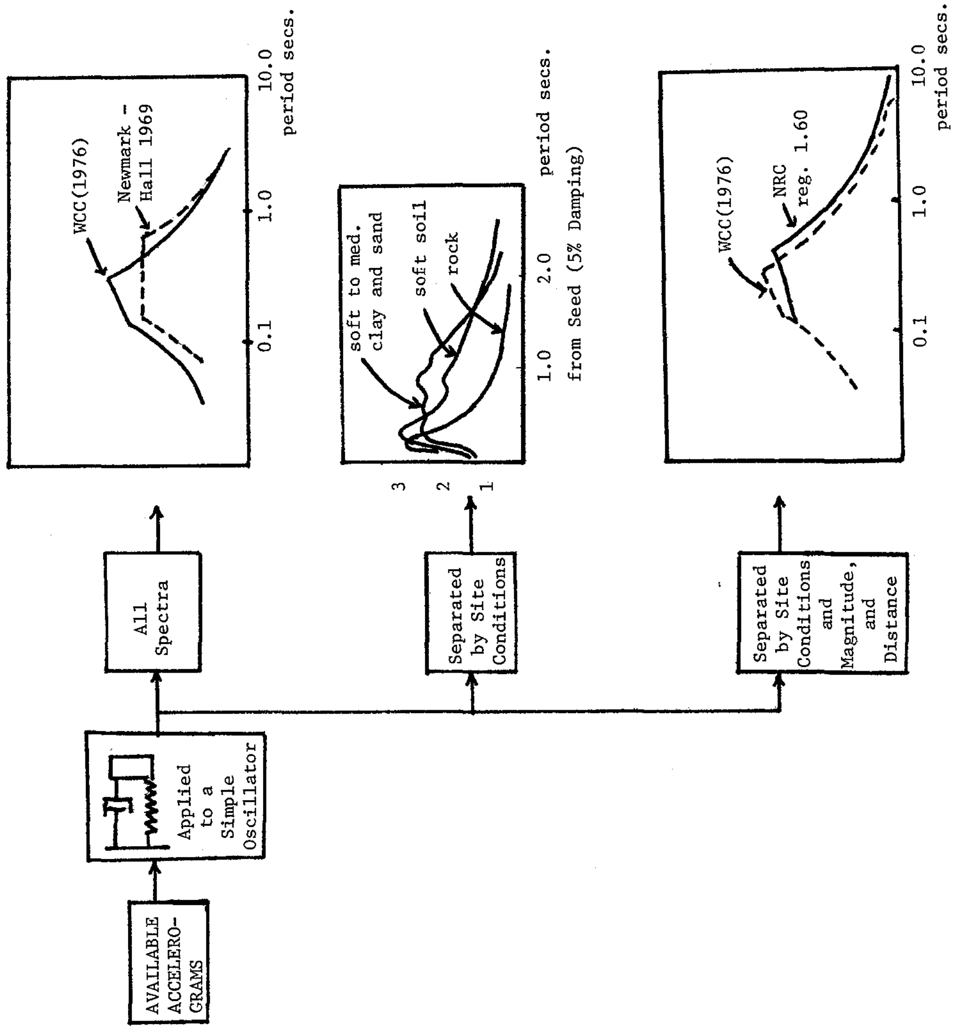
- Most of these maps are usually based on historical data. This data base is extremely short for a reliable future projection. Many geologists have justifiably shown their apprehension in using these maps. Such a problem can be partially solved by using Bayesian statistical concepts, where historical data and any available geological information can be combined to obtain posterior information about a source and the resulting seismicity of a region or site. Various researchers have used this technique in recent years. The problem here is to implement some of these research results for practical use.

- Another major problem with these maps is the selection of a "loading" parameter. The most commonly used parameter for mapping is the peak acceleration, velocity, or displacement. As most of the researchers working in this field of research know, the peak parameters are not the best parameters to represent the "damage potential" of a given earthquake. Secondly, the peak parameters, being extremes, have a very large scatter, resulting in very unreliable estimates and projections. They may give an indication of relative seismic risk, but their mapping (use) for design purposes has led to the perennial arguments between engineers on one side and geologists and seismologists on the other. Recently, many engineers have suggested that "risk maps" which take into account the duration as well as energy content of input and response would be better for representing the "punch" of an earchquake. This better estimation could result in designs which would be consistent with actual risk, rather than perceived or statistical risk. Again, more will be said about this in the next section.

- The third shortcoming of these maps is that the attenuation relationship for peak parameters is extremely poor in a statistical sense. A better parameter would be root mean square (RMS) parameter, this being a statistical average, in a sense, with lower scatter and hence lower uncertainty.

There are various other minor objections to using these maps, but the basic problems are outlined above.

In continuing the discussion of how the seismic risk should be represented, according to the current state of the art, one should direct one's attention to representation of response spectra. There is one school of thought which thinks that a detailed micro-analysis of a site, together with the information on peak input parameter, would give a site-dependent "design" response spectrum. There is another school of thought which looks at all the past available spectral shapes (see Fig. 4). These available spectra are then classified according to site conditions and distance from the source, and a statistical summary of the shape is obtained. An appropriate design spectrum is then selected by multiplying the mean plus $K\sigma$ spectral shape by the appropriate peak ground acceleration. This method, when used with proper and appropriate data, does provide a rational design spectrum with the desired level of uncertainty and risk. However, there are quite a few problems associated with this method of generating probabilistic spectra. First, the method of normalizing the response spectra and then looking at the statistics of the normalized shapes is not very good. This approach forces the response spectra to have zero uncertainty at zero period and then to "wag" the whole spectral shape above this zero period unit


AVAILABLE ACCELERO-GRAMS
Applied to a Simple Oscillator
All Spectra
Separated by Site Conditions
Separated by Site Conditions and Magnitude, and Distance
WCC(1976)
Newmark - Hall 1969
0.1
1.0
10.0
period secs.
soft to med. clay and sand
soft soil
rock
3
2
1
1.0
2.0
period secs.
from Seed (5% Damping)
WCC(1976)
NRC reg. 1.60
0.1
1.0
10.0
period secs.

ordinate to obtain uncertainties at various other periods. Second, availability of appropriate and sufficient data is subject to doubts. Third, there is always the bias in the available data due to the predominance of only certain earthquakes. For example, the San Fernando earchquake data always bias the statistics towards the San Fernando type of earthquake experience. Consider, for example, the spectral ordinate S_a. If o ne wishes to determine the probabilistic information $P[S_a]$, then the conditional probability relation gives

$$P[S_a \text{ and } M_i] = P[S_a/M_i]\ P[M_i] \tag{1}$$

where M_i is an individual event. Summing over all events on all sources within a given area,

$$P[S_a] = \sum_{\text{all } M_i} P[S_a/M_i]\ P[M_i] \tag{2}$$

Thus, most of the statistical spectra available in the literature are actually $P[S_a/M_i]$ since M_i (such as the San Fernando) biases the S_a. What one actually needs is $P[S_a]$ given by Equation (2) above. Equation (2) represents a distinct improvement over current procedures for derivation of probabilistic spectra: the conditional probabilities of spectral shape are derived given the occurrence of an individual event M_i and the marginal probabilities of spectral shape are explicitly obtained based on the probability of occurrence over the active range of events. It presents a possible bias generated by a fixed sample or arbitrarily chosen records. The fourth disadvantages is that there is no guarantee that the total spectral shape has the same probability of exceedence. Thus, it cannot be shown that, for a 95% nonexceedence spectrum, the probability of exceeding S_a at $T = T_1$ is the same as the probability of exceeding S_a at $T = T_2$ and that both these values have only a 5% chance of exceedence.

A recent approach in risk analysis to obtain a probabilistic spectrum is by using attenuation relationships (obtained through regression analysis) for various spectral ordinates at various periods, and using them directly to obtain the mean and the variance of the spectrum (see Figure 5). This method eliminates the arbitrary method of normalizing the available spectrum. However, the method in which the probabilistic spectrum is obtained (using regression analysis - based on sufficient data) can be improved substantially.

The final "chapter" in describing the methodology of risk analysis in the current state of knowledge is to "map" the probabilistic peak ground-motion parameters and probabilistic spectra into a rational design level which would give desired safety at an acceptable cost. This part of the story is very difficult to sort out in the literature, and currently no "universally" accepted simple procedure or formulation exists which can give a design level for an acceptable level of risk.

The above process, as currently perceived by engineers and risk analysis, constitutes the so-called "seismic risk analysis procedure." There are some variations to this theme, but the general ideas are described above. Having explained all that, the question arises, "Where do we go from here?" Do we need more research to add more reports to already-existing (and mostly not

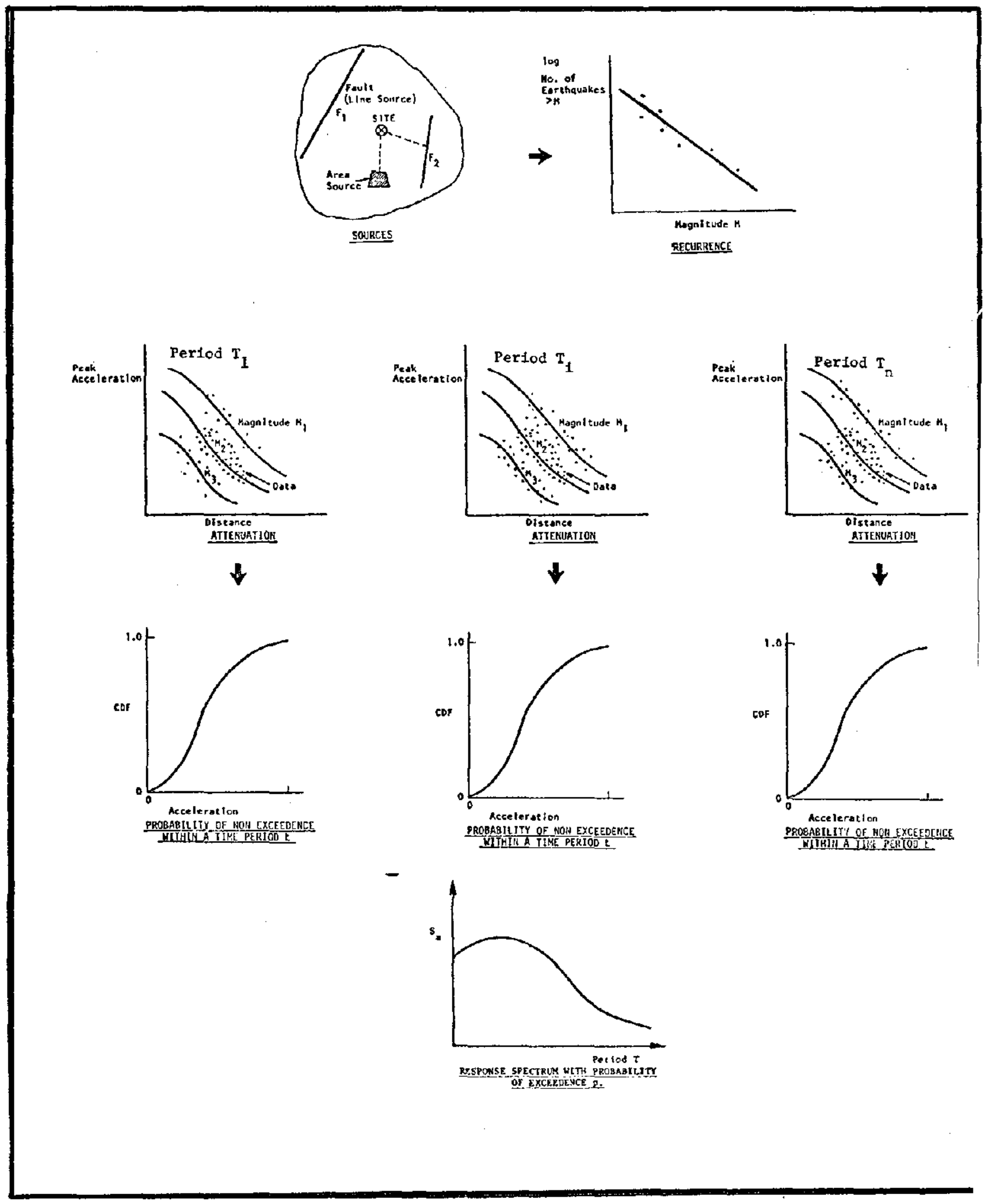


Figure 5. Frequency Content - Current Procedures

practically applied) volumes of reports? Do we need some synthesis of what is available in a simple and practical manner so that a practical "risk analysis" based design methodology can be developed? In the opinion of the authors of this proposal, both tasks are important and necessary. The next section will describe the scope and the direction of the proposed research.

II. RESEARCH NEEDS

The authors are very familiar with the current procedures listed above and have applied them to various regions of the world, including California, Nicaragua, Costa Rica, Guatemala, and Algeria. Most of these available procedures concentrate on defining seismic risk in terms of peak parameters such as peak ground acceleration, velocity and displacement in time domain, and peak spectral response (acceleration, velocity, or displacement) in the frequency domain. It has been felt by engineers and researchers that these peak values do not represent the best indication of the damage potential of an earthquake and also do not lend themselves to a convenient statistical treatment. From past experience of the authors of this paper, it is felt that a better and more stable procedure is needed to refine the accuracy of the results and to provide a better methodology to the user. Of particular importance is the need for providing the user with "equal risk" spectra, i.e., spectra having the same probability of nonexceedence for all period ranges of interest and providing "risk" maps showing the distribution of duration with a selected probability of nonexceedence. Thus, in the total risk-analysis methodology, supplemental procedures which offer the following desirable modifications are the most useful:

1. A better approach for estimating the duration of expected ground motion (input) and a better definition of response duration.

2. A more stable parameter to represent the input ground motion amplitude such as RMS value (e.g., RMS acceleration).

3. A more stable statistical parameter to represent the frequency content (e.g., RMS of the response of a single degree of freedom system for a given damping).

4. Better statistical models that take into account specific geologic and seismologic conditions in the area, such as seismic moments, surface waves, and significant distance.

5. A consistent probabilistic approach at each level of the methodology. Whenever necessary, use of Bayesian statistical methodology to supplement insufficient data.

6. A clear understanding of the needs of structural designers, and the objectives of building codes and their relationship with the probabilistic information on the seismic environment.

The following sections describe the uncertainties and need for research in the above-mentioned six modifications.

A. Input and Response Duration

It is felt that this parameter is one of the most important measures of the damage-producing capability of an earthquake. Some recent studies on input duration have been done by Trifunac and Brady (1975), by Dobry et al. (1977), and by Bolt (1973). There are two definitions of duration needed.

- Duration of input strong motion.
- Duration of response of a single degree of freedom system with a given damping and a period subjected to a given accelerogram.

If one wishes to calculate the energy or the RMS of the input accelerogram, knowledge of the input duration is essential. Similarly, if one wishes to determine the RMS response (or energy in a response) of a single degree of freedom system with a given damping ratio and period, the knowledge about the response duration is required. Fig. 6 shows a typical acceleration response due to a given accelerogram input. It can be seen that the duration is a function of the period of the oscillator, damping, and the input duration. Two definitions of input duration appear to be useful:

- Duration at a particular frequency is the elapsed time between the first and last acceleration excursions greater than a given level (say 0.05 or 0.02 G). Bolt calls this interval the "bracketed duration." It is sometimes measured by cumulatively adding the squared accelerations and adopting the 95 percentile time interval (Husid et al., 1969). However, particularly for earthquakes with a complex multiple source (Wyss and Brune, 1967), this definition often leads to a non-physical upper estimate. Trifunac (1976) and Dobry et al. (1977) adopt this type of approach to obtain duration of earthquake records.
- Duration at a particular frequency is the total time for which acceleration at that frequency exceeds a given value. This interval, called "uniform duration" by Bolt, may equal the corresponding "bracketed duration" or be much less. Uniform duration appears to have a greater mechanical significance with respect to actual structural response behavior.

Working with all the above definitions of earthquake input duration, it is the belief of the authors of this paper that further work is needed. It is proposed that the duration (or equivalent duration) be tied in with the rate of energy arrival as well as the level of energy. Mortgat (1977) conducted a sensitivity study of the effects of various levels of acceleration cutoffs on the duration. He arbitrarily decided to use .02g level of acceleration as the cutoff acceleration. If one follows the method suggested by Trifunac & Brady in which the equivalent duration is that time during which 5% to 95% of the energy arrives (i.e., $D_{95\%} - D_{5\%}$ = equiv. duration), the correlation between that definition and Mortgat's definition is very low. However, duration of a <u>strong motion</u> cannot be defined based only on the time during which 95% of the energy comes, but should also depend on the <u>level</u> of that energy. Suffice it to say that, since input duration has a direct bearing on the parameters such as the RMS and the energy and eventually affects the damage potential, a clear

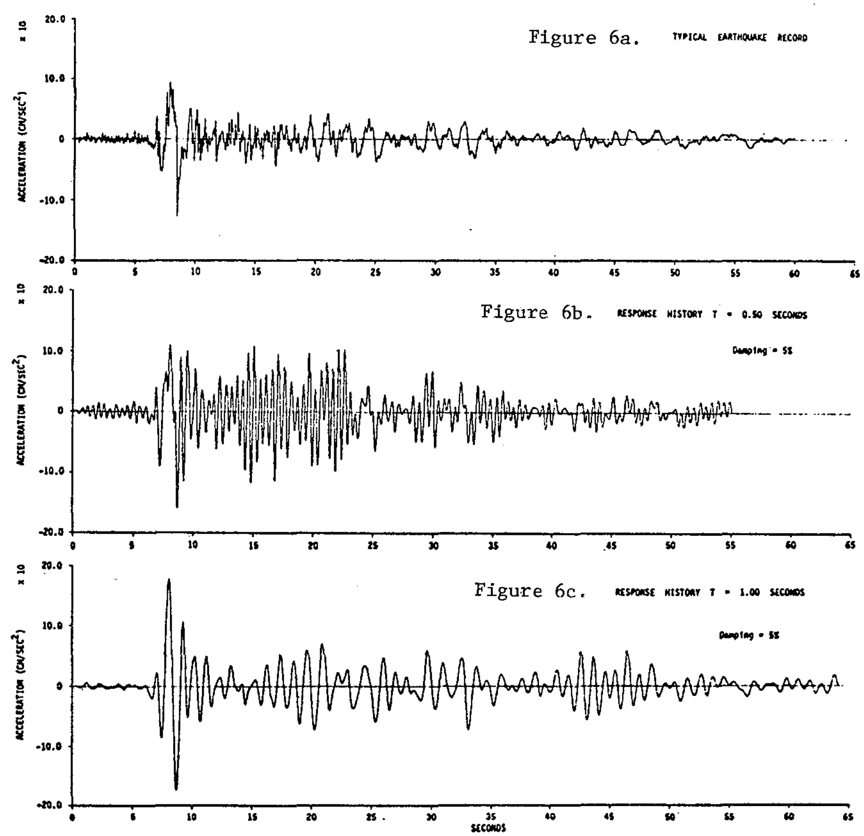


Figure 6. Typical Earthquake Record and Response Time Histories

and simple definition is needed. Hence, the first task of a future research program should be to look at the "input duration."

Surprisingly, no work is available in the literature defining the duration of the response record. Unless a convention (or a standard procedure) is used to define this quality, it is not possible to evaluate parameters such as the RMS of the response, energy in the response, number of peaks in the response, etc. Mortgat arbitrarily defined this quality as follows:

> "The response duration is obtained by terminating the response when the amplitude of response acceleration peak reaches 10% of the highese response peak and does not exceed that value thereafter."

Naturally, more can be done to improve such a definition. Thus, the second task of a future research program should be to investigate the definition of response duration.

B. Stable Statistical Parameters for Amplitude and Frequency

As mentioned in the introduction, using the peak ground motion parameter to represent the "punch" or the "loading" or the "damage potential" of an earthquake may not be the best. Various researchers have recommended that some other parameter, such as the RMS or energy, be used to represent the "loading" at a given site. The peak parameter, being an extreme, has considerable uncertainty associated with it. Thus, if a parameter such as anRMS, which is a statistical moment of sorts, is used, the uncertainty in the model could be reduced. Thus, an earthquake input could be represented in the time domain by either an RMS or an energy, instead of its peak parameter value. The frequency domain characteristic could be represented by the RMS acceleration (as an example) as a function of the period of the oscillator, instead of the maximum response peak such as a spectral acceleration (S_a).

The selection of stable statistical parameters is based on statistical analysis currently under way at Stanford University on the characteristics of 97 time histories recorded around the world and their response spectra developed by applying these time histories to a single degree of freedom system with damping ratio β and natural period T. A typical case is shown in Fig. 6. Fig. 6a shows a typical recorded time history. Figs. 6b and 6c show the response time history relationships for damping of $\beta = 5\%$ and periods $T = 0.5$ sec and 1.0 sec, respectively. Figs. 7a and 7a show peak and RMS acceleration spectra for two typical cases. The RMS values were derived from the response time curves using the equation

$$\text{RMS} = \sqrt{\frac{h_i^2}{n-1}}$$

where h_i is individual peak amplitude and n is the total number of peaks.

A comparison of peak and RMS spectra shows that in all cases RMS is a more stable indicator of the earthquake motion. Moreover, from a probabilistic point of view, RMS behavior can be more easily modeled than peak behavior. RMS is based on sufficient statistics, and the tail of the distribution, often inaccurate, is not governing as it is in the case of peak modeling. Peak amplitudes are often the chance result of a probabilistic transient phenomenon.

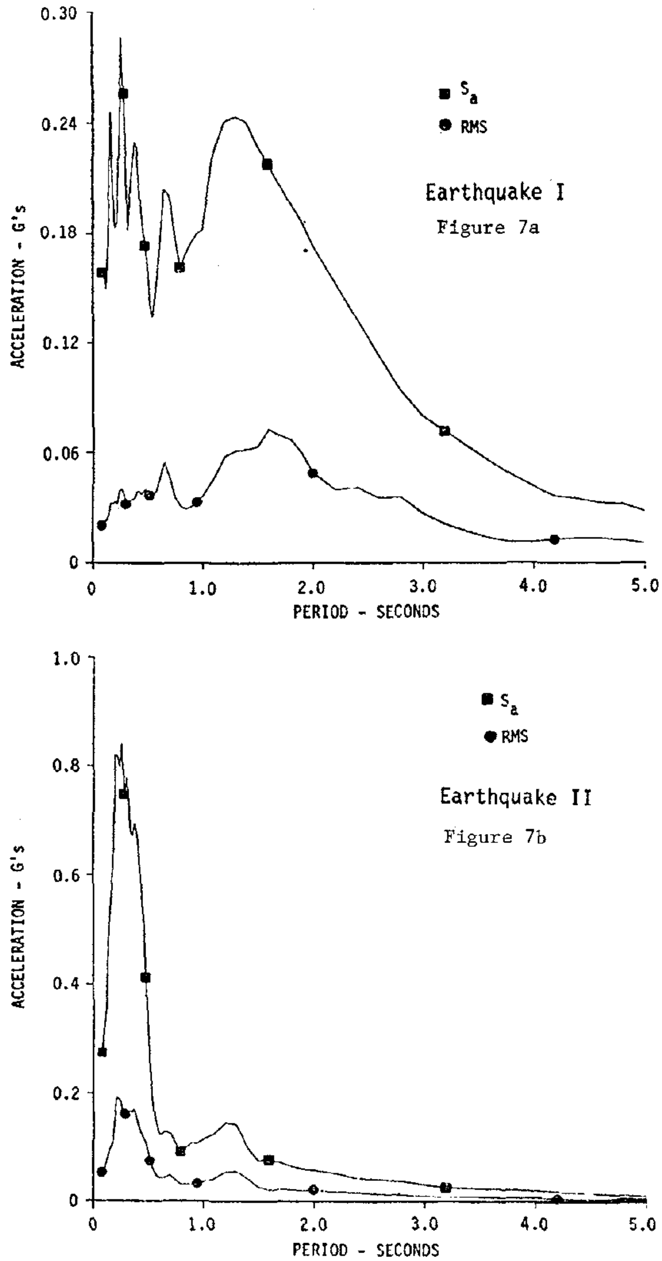


Figure 7. TYPICAL ACCELERATION AND RMS SPECTRUM

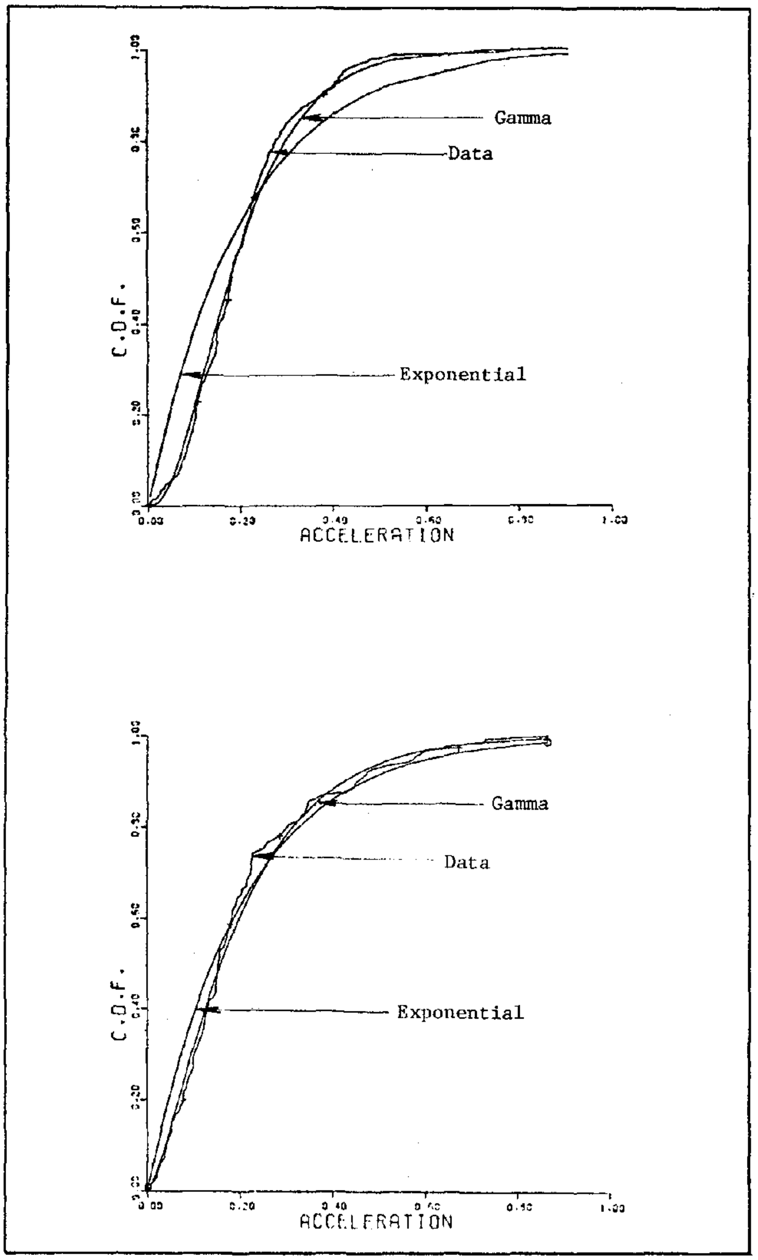


Figure 8. Two Typical Gamma and Exponential Fits of Earthquake Record Amplitudes.

They show great scatter even for events considered similar (same distance, magnitude, and site conditions). A better and more stable parameter based on sufficient statistics rather than extreme values provides a better definition of the amplitude content of ground motion.

Furthermore, use of the peak value provides information on maximum amplitudes without giving any information about the other peaks, whereas parameters such as mean or RMS filter out information about the whole input and implicitly include information on all the peaks and their distribution.

Using response-time relationship, shown in Fig. 7b, a distribution is fitted considering the amplitude of each peak as a random variable. A Gamma distribution fits the data very well for all earthquakes, for all damping ratios (β varied from 5 to 20 percent) and periods ($T = 0.05$ sec to 5 sec). The parameters of the distribution depend on the individual event and the period. The remarkable agreement between the computed CDF and a theoretical Gamma as well as exponential function is shown in Fig. 8. Similar statements can also be made for input accelerogram peaks.

Assuming that the distribution of peaks (input or response) is exponential, then the acceleration a_p, which could be input ground motion ($T = 0$) or the response acceleration ($T = T_i$) which has p percent chance of exceedence, can be obtained. Fig. 9 shows the exponential distribution and its properties.

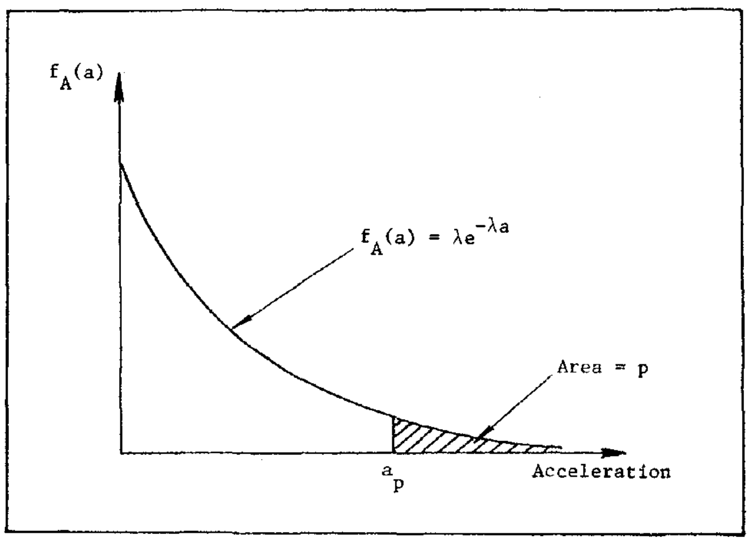


Figure 9. Exponential Distribution

The RMS value for this distribution is given by

$$\mathrm{RMS} = \frac{\sqrt{2}}{\lambda}$$

Considering the acceleration a_p which has $p\%$ chance of being exceeded

$$e^{-\lambda a_p} = p$$

or

$$a_p = \ell n \left(\frac{1}{p}\right)$$

Hence

$$a_p = \frac{1}{\lambda} \ell n \left(\frac{1}{p}\right)$$

and the ratio

$$K_1 = \frac{a_p}{\mathrm{RMS}}$$

$$= \frac{\ell n\left(\frac{1}{p}\right)}{\sqrt{2}}$$

From the above equation, it can be seen that the ratio $K_1 = a_p/\mathrm{RMS}$ depends only on p and is independent of λ or the individual peaks. Thus, for $p = .05$, $K_1 = 2.12$, and for $p = .10$, $K_1 = 1.63$. Fig. 10 shows the remarkable agreement between these theoretical values and the actual values obtained by analyzing about 97 responses. It can also be seen from Fig. 10 that the uncertainty in this value of K_1 is very small. Having shown that $K_1 = a_p/\mathrm{RMS}$ is practically a constant for all periods for all recorded earthquakes, it was further observed that the value of K_p varies insignificantly with damping (see Fig. 11). Since the constant behavior of K_1 is interpreted from an analysis of 97 time histories recorded over widely different magnitudes, distances, site conditions, and transmission path conditions, it can be considered to be a truly stable statistical parameter. The constancy of K_1 has also been verified for the accelerograms themselves (for $T = 0$).

The remarkable stability of K_1 and the relative stability of RMS (Figs. 7a and 7b) can be utilized for deriving response spectra that can be considered as "equal risk spectra", i.e., response spectra with spectral values having the same probability of nonexceedence for all period bands (Fig. 12). The advantages of such a spectrum to a designer are readily apparent when compared with spectra such as those shown in Fig. 4, which do not represent the same level of probability of nonexceedence for all periods.

To determine the probability of occurrence of a certain RMS value, one can write the following equation:

$$P(\mathrm{RMS\ and\ } M_i) = P(\mathrm{RMS}/M_i)\ P(M_i) \quad ,$$

where M_i is an individual seismic event. Then

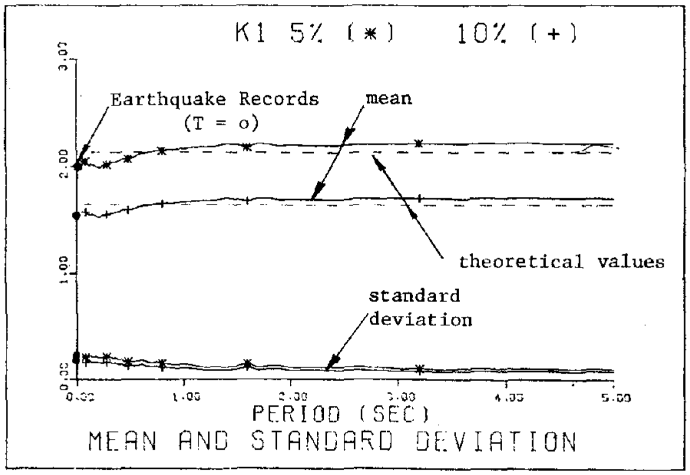


Figure 10. Factor K_1 (Statistics for 97 Responses)

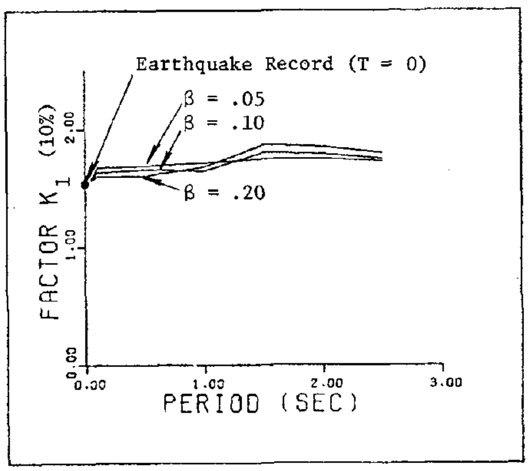


Figure 11. Typical Behavior of Factor K_1 for Different Dampings β.

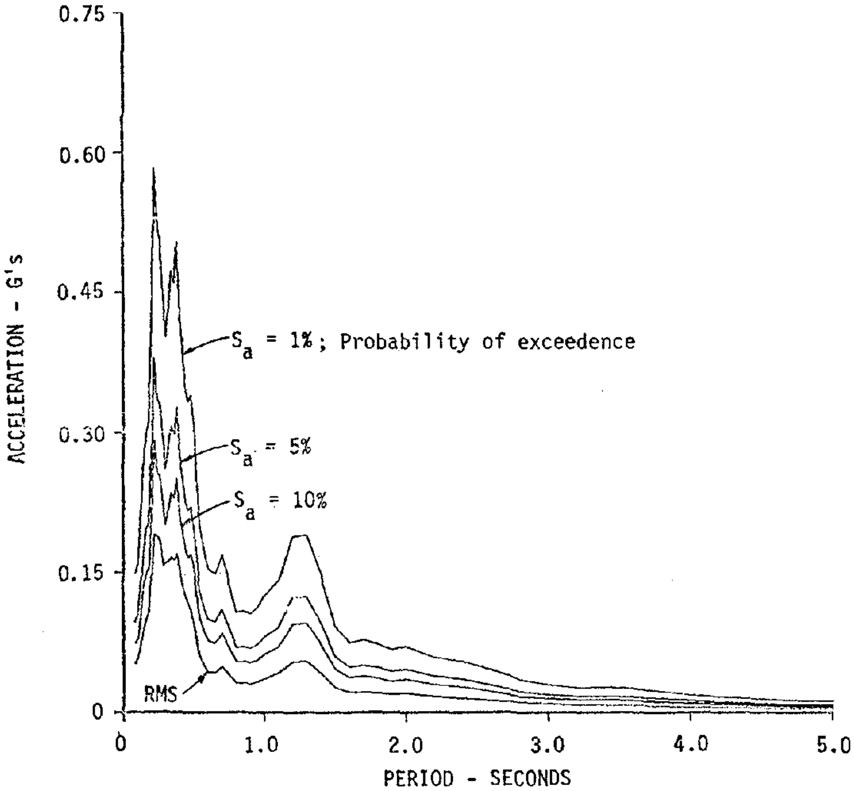


Figure 12. RMS AND ACCELERATION SPECTRA

$$P(RMS) = \sum_{\text{all } M_i} P(RMS/M_i)\ P(M_i)$$

The above equation represents a distinct improvement over current procedures dor derivation of spectra. First, the conditional probabilities of spectral shape are derived given the occurrence of an individual event M_i; and, second, the marginal probabilities of spectral shape are explicitly obtained based on the probability of occurrence over the active range of events. It prevents a possible bias generated by a fixed sample of arbitrarily chosen records.

An acceleration spectrum with a given probability of exceedence can then be obtained taking advantage of the factor K_1.

$$a_p = K_1 \text{ RMS} \quad \text{(for all periods)} \quad ,$$

where

p = probability of exceedence,
a = acceleration corresponding to p,
K_1 = stable parameter corresponding to p.

a_p is not the peak value having a probability p of being exceeded, but rather the acceleration in the whole response history of period T_1 that will be exceeded with a probability p.

Having observed the above behavior, the following steps are considered essential in conducting further research.

<u>Step 1</u>. Investigate on a theoretical basis to see why the parameter $K_1 = a_p/RMS$ does behave in such a stable manner. Is the behavior and value of K_1 dependent on soil condition, period, type of earthquake, etc.?

<u>Step 2</u>. Based on the geological, seismological, and tectonic environment, can one postulate an RMS spectrum shape? A subtask for this work would require RMS attenuation studies for various magnitude earthquakes.

<u>Step 3</u>. With the aid of RMS response spectra and the stable parameter K_1, how can one obtain "design spectra"? One way to look at this problem is to select an RMS response spectrum and a probability (or risk) of exceedence p. Then $(a_p) = (RMS)_T\ (K_1)_p$. One question that needs to be answered is, "Which RMS spectrum should be chosen -- mean RMS spectrum or mean + no RMS spectrum? Next, if the chosen spectrum for RMS is the mean spectrum and the desired non-exceedence probability of the response for all periods is p, then what value of K_1 should be selected? Thus, in this task, a detailed look at combining the probabilistic information on RMS spectra and the response spectrum is needed.

<u>Step 4</u>. A very interesting parameter to work with is the energy of the input or the response. If one defines

$$K_2 = \frac{(RMS)^2\ T^2}{(ENGY/NBPK)}$$

where

RMS = root mean square,
T = period of the oscillator,

ENGY = cumulative potential energy stored in the spring of the oscillator,

NBPK = number of zero crossings of the given time history.

Thus, K_2 is a function of energy as well as RMS. From the current study at Stanford, this parameter is also very stable for all the 97 accelerograms studied. This step should examine this parameter K_2 and see whether the energy-related parameter can help in developing a definition of duration. It is not possible to describe all the possible ramifications in this short paper, but it can be said that there are various interesting possibilities to look at in such a study.

Step 5. Having studied the probabilistic description and the behavior of input and response from past data, what mapping or hazard-zoning parameters should be used? This study should involve defining mapping parameters and associated probabilities. Mapping parameters should consist of spectral values for selected levels of nonexceedence amplitudes, such as peak acceleration velocity or displacement and duration for a selected level of nonexceedence. To derive the spectral values, an RMS spectrum should be developed as an intermediate step.

III. CONCLUSION

This paper presents some of the uncertainties associated with seismic ground motion. Major shortcomings of the presently used methodologies for seismic-hazard mapping are pointed out, and improvements are proposed to minimize some of them.

Specific results of work presently done on RMS and attenuation relationship will be presented at the workshop.

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AN OVERVIEW OF THE STATE-OF-THE-PRACTICE IN EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION

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EVOLUTION OF CODES AND STANDARDS FOR EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)

by

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The need for ductility in the lateral force-resisting systems of buildings, that is the capability to continue to absorb energy while deforming inelastically without substaining excessive damage, has been recognized by earthquake engineers for many years. Extensive research and study have been performed by numerous academic, industrial and engineering groups over the past two or more decades in attempts to improve analysis, design, and construction procedures so as to obtain more earthquake-resistant buildings. A perspective of how seismic codes have developed provisions for earthquake-resistant concrete buildings can best be gained from a review of the activities of the Structural Engineers Association of California (SEAOC), the Portland Cement Association (PCA), Applied Technology Council (ATC), and others. The problems encountered in introducing research results into codes and implementing them in practice are then discussed, followed by a listing of problems in ERCBC that remain to be resolved.

1959 SEAOC BLUEBOOK

The "Recommended Lateral Force Requirements" (Bluebook) [1], published by SEAOC in 1959, presented several new approaches to earthquake-resistant design which tended to give bonuses for ductile designs. The bonuses were in the form of lower seismic factors. These same provisions or variations thereof have been included in almost all seismic codes since 1959.

The fundamental period of the building, T, was introduced into the calculation of the seismic lateral force base shear, V, by the formula:

$$V = KCW$$

where K = horizontal force factor varying from 0.67 to 1.33

$$C = \frac{0.05}{\sqrt[3]{T}}$$

T could be determined using recognized and substantiated methods or could be calculated by the formula:

$$T = \frac{0.05\ H}{\sqrt{D}}$$

where H was the height of the main portion of the building and D was the dimension of the building in the direction of the applied forces.

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However, for all buildings where the lateral force-resisting system consisted of a moment-resisting space frame (MRSF) which resisted 100 percent of the required lateral force and was not enclosed by or adjoined by more rigid elements which would tend to prevent the frame from resisting lateral forces, T = 0.10N, where N was the total number of stories above grade. For most buildings, this formula gives a longer period and hence lower values of C.

It should be noted that in 1952 a Joint Committee on Lateral Forces of the San Francisco Section, ASCE, and the Structural Engineers Association of Northern California first recommended the use of building period to determine C (the coefficient C = K/T, where K equalled 0.015 for buildings and 0.025 for other structures). The period T could be calculated or estimated. In 1956, San Francisco adopted a variation of the Joint Committee recommendations.

Another important provision in the 1959 Bluebook was the introduction of the idea of varying the horizontal force factor, K, dependent on type of structure and type of construction (see Table 1). Buildings with moment-resisting space frames were assigned the lowest value of K equal to 0.67. Buildings with bearing wall (shear wall) box systems were assigned K values of 1.33. Again the result is a bonus for ductile construction.

Special restrictions for tall buildings were included in Section 2312 (j):

> (j) Structural Frame. Buildings more than 13 stories or one hundred and sixty feet (160') in height shall have complete moment resisting space frames capable of resisting not less than 25 percent of the required seismic load for the structure as a whole. The frame shall be made of a ductile material or a ductile combination of materials. The necessary ductility shall be considered to be provided by a steel frame with moment resistant connections or by other systems proven by tests and studies to provide equivalent energy absorption.

The specific requirement that ductile moment-resisting space frames (DMRSF) must be provided in buildings over 13 stories or 160 feet in height was an added impetus to research on how to obtain ductility in concrete frames. The last sentence specifically required tests and studies be made for materials other than structural steel to prove equivalent energy absorption.

The Commentary to the Bluebook was published in 1960 and contained important clarifications and discussions relating to the intent of the provisions and requirements for reinforced concrete (RC). The general intent was spelled out:

> The code does not assure protection against nonstructural damage such as cracked plaster, broken glass, broken light fixtures, cracked ornamentation, cracked filler walls, or overturned equipment. Neither does it assure protection against all structural damage. It is pointed toward confining structural

damage to minor, repairable damage that would not jeopardize the safety of the structure.

In the discussion on details, requirements for masonry including concrete, were given:

> All masonry, including concrete, which is considered to be part of the structural system resisting lateral forces, is required to be reinforced. All other masonry should be kept free of the structural system so that participating stresses are not introduced which will contribute to the failure of these nonstructural elements. Otherwise they too should be reinforced. Beams and columns or piers of reinforced concrete earthquake-resisting elements should be provided with stirrups to resist the full computed combined shear due to vertical and lateral loading, independent of the shear resistance of the concrete.

The last sentence was implicit recognition that RC structures should be designed so as not to fail in shear.

Prior codes, such as the 1952 UBC [2], contained a few special seismic requirements for RC such as interconnection of pile caps or foundations on poor soil. The interconnecting ties were required to be able to transmit 10 percent of the total vertical load on the heavier of the footings of foundations connected. The minimum size of ties was specified together with minimum reinforcement. Reinforced concrete slabs were permitted in lieu of the tie members providing adequate reinforcing and connections to the footings were provided.

PCA MANUAL - 1961

About the same time the 1959-60 SEAOC Bluebook was being prepared, the PCA undertook development of a design manual, "Design of Multistory Reinforced Concrete Buildings for Earthquake Motions" [3]. The manual, by Blume, Newmark and Corning, presented information on earthquake motions, dynamic behavior, behavior of structures, principles of earthquake-resistant design, seismic design codes and specifications, behavior of RC members under static and dynamic loads, design recommendations to provide the necessary ductility in RC buildings, illustrative design examples, and recommendations for construction procedures and inspection.

Development of the manual was based on careful review and evaluation of the work of many authors, investigators, and technical committees [4-28]. Available research results on RC together with knowledge of structural response to earthquake motions were blended and translated into design procedures and recommendations. Design, construction, and inspection requirements necessary to ensure ductility in RC members were developed. The manual became an excellent reference work and was used, together with results of a number of research projects conducted by or for PCA on beam-column connections and RC shear walls, by the SEAOC Seismology Committee to develop the 1966 Revision of the SEAOC Bluebook [29].

TABLE 1
(from 1959 SEAOC BLUEBOOK, Ref. 1)

HORIZONTAL FORCE FACTOR "K" FOR BUILDINGS OR OTHER STRUCTURES[2]

Type or Arrangement of Resisting Elements	Value of K[1]
All building framing systems except as hereinafter classified.	1.00
Buildings with a box system as defined in Section 2312(b).	1.33
Buildings with a complete horizontal bracing system capable of resisting all lateral forces, which system includes a moment resisting space frame which, when assumed to act independently, is capable of resisting a minimum of 25% of the total required lateral force.	0.80
Buildings with a moment resisting space frame which when assumed to act independently of any other more rigid elements is capable of resisting 100% of the total required lateral forces in the frame alone.	0.67
Structures other than buildings and other than those listed in Table 23-D.	1.50

([1]) The coefficients determined here are for use in the State of California and in other areas of similar earthquake activity. For areas of different activity, the coefficient may be modified by the building official upon advice of seismologists and structural engineers specializing in aseismic design.

([2]) Where wind load as set forth in Section 2307 would produce higher stresses, this load shall be used in lieu of the loads resulting from earthquake forces.

The PCA manual recommended a number of significant additions to the requirements for RC specified in ACI 318-56 [27]. The recommendations were developed to ensure that a minimum ductility factor corresponding to $\mu = 4$ could be achieved without loss in strength. It was recognized that additional ductility and energy absorption would be available beyond this value of μ, although minor damage such as spalling might occur.

It was felt that the recommendations presented were sufficient for RC structures to have the ductility and strength required to resist major earthquakes. A number of important points that are basic to designing RC frames and structures for ductility were listed:

1. Transverse or shear reinforcement should be provided to make the strength in shear greater than the ultimate strength in flexure.

2. The amount of tensile reinforcement should be limited, and/or compression reinforcement used to increase energy absorbing capacity.

3. Critical sections of stress concentrations, such as column-beam or column-girder connections, should be confined by hoops or spirals so as to increase the ductility of the columns under combined axial load and flexure.

4. Splices in reinforcement should be given special attention and care should be taken to avoid planes of weakness that might be caused by bending or terminating all bars at the same section.

Beams and Girders - Detailed specifications were presented for the amount of longitudinal reinforcement in beams at connections to columns, and minimum reinforcement both top and bottom of beams for their entire length (needed to resist reversals in bending moments). Web reinforcement was specified for beams and girders in frames to ensure that their capacity was governed by flexure and not by shear. Where reinforcing could act as compression reinforcement, stirrup-ties were required to restrain the bars from buckling after spalling of the concrete cover.

A number of other new requirements were recommended. Because the peak bending moments along a girder can vary both in sign and location during a severe earthquake, cut-off points of bars were specified and the use of straight rather than bent-up bars was recommended (see Figure 1). Bar anchorage was specified to ensure that bars would not pull out despite being subjected to a large range of deformations.

Columns - Full confinement of concrete in columns at beam-column connections was recommended except where the maximum axial compressive stress on gross column area expected during an earthquake would not exceed 12 percent of the concrete compressive strength. Vertical reinforcement ratio in columns should be at least 1.0 percent with a maximum of 6 percent. Figures 2, 3, and 4 show typical transverse reinforcement requirements. During an earthquake, columns are expected to resist large bending moments and may be subjected to tensile stresses; therefore, special splicing considerations are necessary (see Figure 5).

Walls - The requirements of ACI 318-56 were followed with the added recommendation of special reinforcing at corners and junctions (see Figure 6). Supplementary reinforcing was suggested at wall openings and horizontal and vertical construction joints (see Figure 7). Splices in adjacent reinforcing bars should be staggered at least 18 inches.

The recommendations presented in the PCA Manual provided a major basis for the development of code provisions for ductile RC buildings.

1966 SEAOC BLUEBOOK [29]

The next major advance in code requirements for ERCDC was the publication of the 1966 Revision to the SEAOC Bluebook. In the period from 1960 to 1965, the 13-story limit was deleted in 1963, but the 160-foot height limit was retained.

Detailed specifications for RC ductile moment-resisting frames were presented in the 1966 Revision. The details were developed from proposals submitted by PCA [30-34] and a special study for SEAOC [35]. The requirements were in addition to those specified in ACI 318-63.

The 1966 revisions were directed to qualifying RC moment-resisting space frames to the necessary ductility for buildings more than 160 feet in height. Normally it is desirable to write performance-type code requirements; however, because complete information was not available, detail specifications were prepared. The intent was to prevent brittle modes of failure by ensuring that the tensile reinforcing would yield prior to compression, shear, or anchorage failures. Based on observed performance of structures in severe earthquakes, the specifications were developed so that ductile behavior should occur at all levels of load and deformation even beyond yield and for reversal of stresses.

Requirements for the use of shear walls or braced frames in conjunction with ductile moment-resisting frames, and design, construction, and inspection provisions were included. A distinction was made in the definitions of Moment-Resisting Space Frames (MRSF) and Ductile Moment-Resisting Space Frames (DMRSF). A new provision was added requiring that all buildings designed with a horizontal force factor K, of 0.67 or 0.80 shall have DMRSF. It was further specified that shear walls in buildings where K = 0.80 shall be composed of axially-loaded bracing members of structural steel, or RC bracing members or walls conforming with the specified requirements. Reinforced concrete shear walls and RC braced frames for all buildings were also to conform to the requirements given.

Physical Requirements - A minimum concrete strength of 3,000 psi was required. Beam and girder reinforcing was limited to ASTM A-15, A-408, or A-432, with the A-15 or A-408 either structural or intermediate grade with the specified yield strength not to exceed 40 ksi. For columns, the specified yield strength for vertical steel was not to exceed 60 ksi.

Flexural Members - A minimum width-depth ratio of 0.4 with minimum width of 10 inches or the supporting column width plus a distance equal to three-fourths of the depth of the flexural member on each side of the

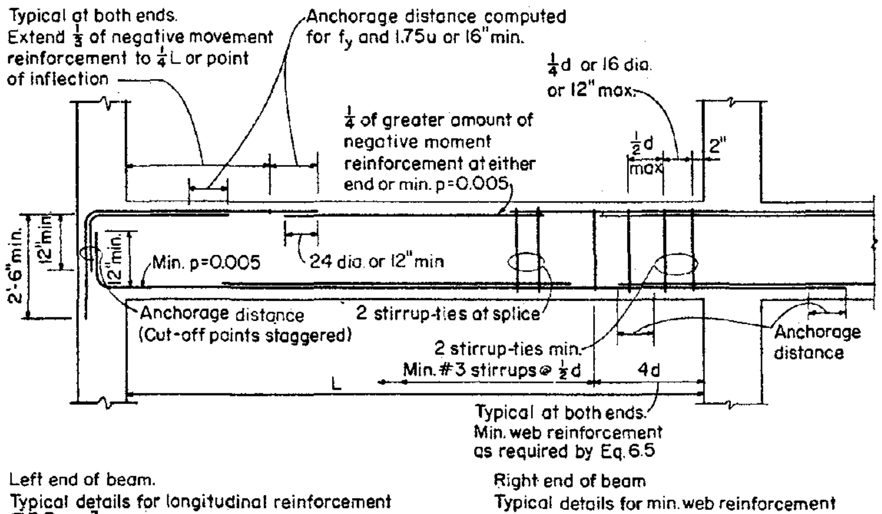


FIG. 1

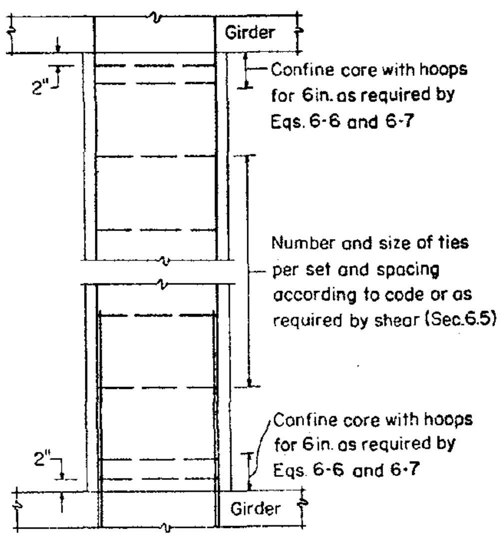
Typical longitudinal and web reinforcement details for girders and beams on column lines.

(from Blume, Newmark and Corning, Ref. 30)

FIG. 2

Elevation of a column showing transverse reinforcement when confinement of the concrete is not required.

(from Blume, Newmark and Corning, Ref. 30)



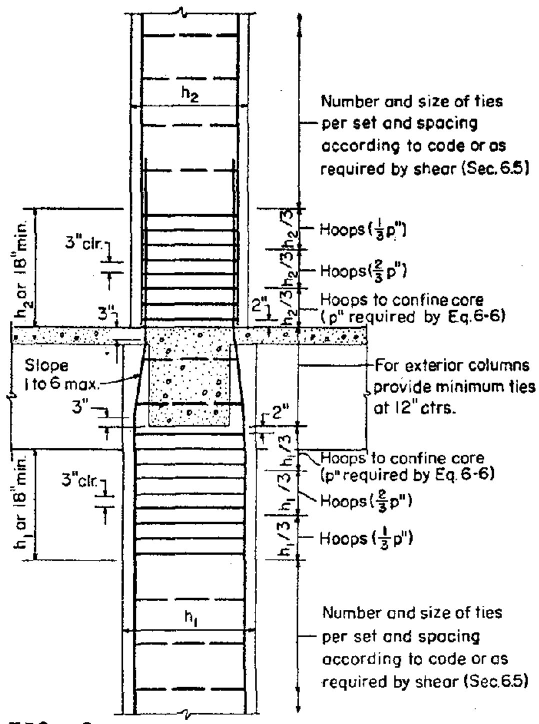


FIG. 3

Elevation of a column showing transverse reinforcement at a joint when confinement of the concrete is required.

(from Blume, Newmark and Corning, Ref. 30)

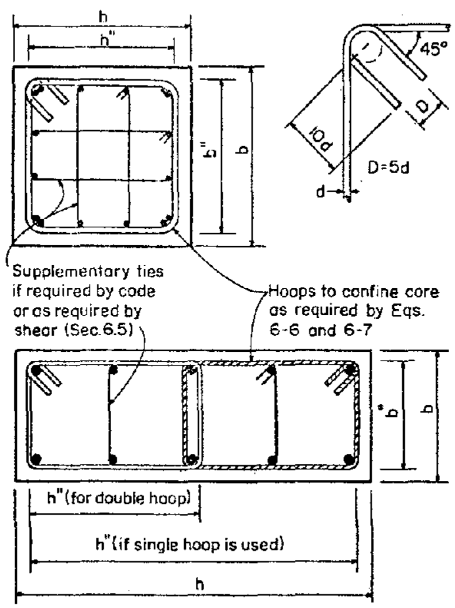


FIG. 4

5. Typical arrangements of transverse reinforcement in square and rectangular columns in which confinement of core is required to increase strength or ductility.

(from Blume, Newmark and Corning, Ref. 30)

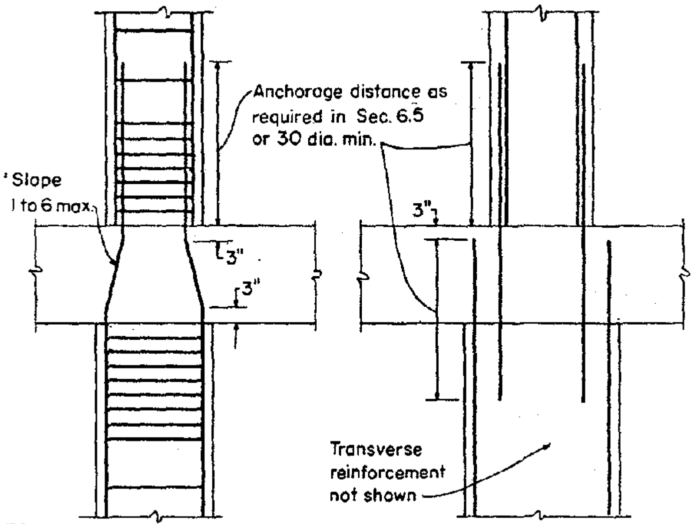


FIG. 5
Elevation of a column showing anchorage requirements for vertical reinforcement and dowels.

(from Blume, Newmark and Corning, Ref. 30)

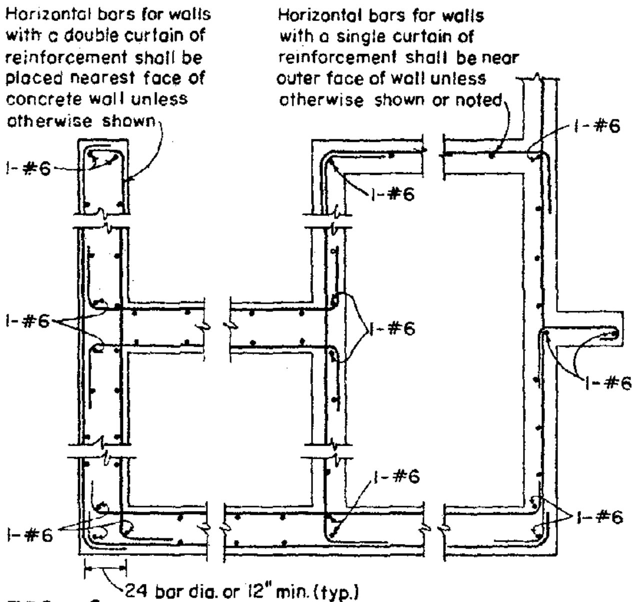
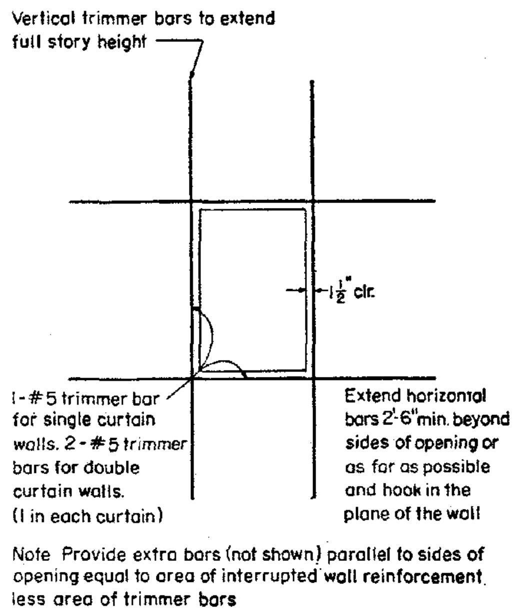
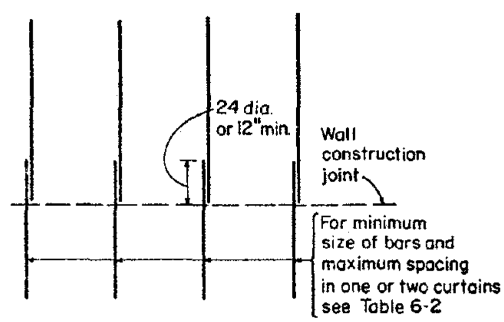


FIG. 6
Plan view showing typical wall reinforcement details.

(from Blume, Newmark and Corning, Ref.30)



a) **Supplementary reinforcement to be provided at wall openings.**



b) **Recommended reinforcement detail at horizontal and vertical wall construction joints.**

FIG. 7 WALL OPENINGS AND CONSTRUCTION JOINTS (from Blume, Newmark and Corning, Ref. 30) column was established. Flexural members shall have a minimum reinforcement ratio, for top and for bottom reinforcement, of $200/f_y$ throughout their length with at least two bars provided both top and bottom. The reinforcing ratio p was limited to 0.025 or $0.46\ f_c'\ p'/f_y\ p$ for negative moment at the column face. In addition, the positive moment capacity at such locations was specified to be at least 50 percent of the negative moment capacity provided. A minimum of 25 percent of the larger negative reinforcement at either end was required full length of the beam.

No splicing of tensile rebar in areas of tension was allowed unless the region was confined with stirrup-ties. No splices were allowed within the column or within 2d from the face of the column.

Anchorage is a very important requirement. At exterior columns, the beam reinforcing was required to be anchored in the confined region of the beam-column joint. Recognition was given in calculating the length of anchorage to the fact that bond strength in confined regions could be increased up to 50 percent.

Web reinforcing was specified in accordance with Chapter 17, ACI 318, except that the shear capacity should equal the vertical load shear plus the shear resulting from the ultimate moments at the ends of the beam. The use of inclined stirrups was prohibited unless it could be shown that the shear stress would not reverse in direction under earthquake loading. A maximum stirrup spacing of d/2 was given to help ensure that unexpected shear failures did not occur within the beam. Shear reinforcement in areas where confinement is required were specified to be stirrup-ties. Stirrup-ties were also required wherever compression reinforcement is needed.

Columns - A dimensional limitation on columns was stipulated to ensure that columns generally conformed to the proportions of those tested. The ratio of minimum to maximum thickness was set at not less than 0.4 with the least column dimension at 12 inches.

The reinforcement ratio for tied columns was specified at one percent minimum and six percent maximum. The six percent limit was set so as to minimize congestion in the column, especially at the beam-column connection.

Splicing of longitudinal reinforcing was permitted only in the center half of the column height so as to avoid locations where inelastic hinging could occur.

Special transverse reinforcement (STR) was specified through the joint plus a distance equal to the maximum column dimension either side of the joint but not less than 18 inches. STR confines the concrete and increases the strain capacity of the concrete within the core, and also inhibits buckling of the longitudinal reinforcing.

The required volume of STR could be satisfied by spirals per ACI 318 Section 913; or if hoops were used, the volume was set at two times the required spiral reinforcement. It was felt that hoops had about 50 percent of the efficiency of spirals. Detail specifications were given as to use of supplementary cross ties, unsupported length of hoop, overlapping of hoops, size, and center-to-center spacing. The volume of STR required could be reduced to one-half where flexural members frame into all four sides of the column.

A beam-column joint analysis was required to check whether the STR provided per the above requirements was adequate to resist the maximum shear developed under ultimate loading conditions. The effects of beams framing into all four sides of the column would reduce this shear requirement. The effective length of the column for design was in accordance with ACI 318, Sections 915(d) and 916.

Sufficient transverse reinforcement in columns subject to bending and axial compression was required to resist the maximum ultimate column shear. Where the design axial compressive stress was less than 12 percent of the concrete compressive strength, the concrete was not relied upon to resist the shear in the column.

In order to ensure the overall vertical stability of the structure during a severe earthquake, columns at beam-column joints were required to have a greater ultimate moment capacity,at the design earthquake axial load, than the ultimate moment capacity of the beams framing into the joint. Where the design axial compressive stress is less than 12 percent of the concrete compressive strength, the column was required to conform to the requirements for flexural members.

The new reinforcing details required special inspection for DMRSF to ensure that the steel and concrete were properly placed. The inspector was required to be specially qualified and under the supervision of the professional engineer responsible for the design. A summary of the 1966 SEAOC code requirements is presented in Table 2.

Concrete Shear Walls and Braced Frames - The 1966 Bluebook also included provisions for the design and construction of RC shear walls and braced frames. The basic requirements of ACI 318 were followed plus several additional requirements. Ultimate strength design was specified with working stress acceptable providing there was an equivalent factor of safety. The load combination equations were the same as specified for DMRSF; however, for buildings without a 100 percent MRSF, the design shear stress level was reduced 50 percent. The concern was to minimize the possibility of brittle behavior or nonductile failure of the shear walls.

The nominal ultimate shear stress in shear walls was limited based on factors proposed by PCA [36]. For short squat walls, the shear capacity of the wall was based on the shear capacity of the concrete. For taller shear walls with H/D greater than 2.7, the shear capacity was set essentially the same as in ACI 318. Reinforcing was to meet the requirements of ACI 318.

Vertical boundary members were specified for buildings with K = 0.80. The boundary elements were provided to act as flanges for the shear wall acting as a vertical cantilever. Horizontal reinforcing was required to be fully anchored to the vertical elements. The vertical elements were required to be sized to resist all vertical dead and live loads plus vertical earthquake overturning loads, and to be confined for their full length. Similar requirements were given for wall openings.

Reinforced concrete braced frames in DMRSF buildings were required to have STR throughout their full length in order to inhibit brittle compressive failures. Tension members in such frames were required to meet the requirements for compression members.

A summary of the 1966 SEAOC code provisions for shear walls and braced frames is given in Table 3.

1967 THROUGH 1974 SEAOC, ATC, AND ACI 318-71

The 1966 SEAOC code was a major change from prior RC requirements. Initially, considerable opposition was expressed by structural engineers, primarily on the basis of the extensive additional design time required and the probable extra construction cost. However, as designers worked with the code and became familiar with its requirements and as contractors became experienced in the many steel congestion problems, the opposition diminished.

During the five years from 1966 to 1971, a few changes were made in the SEAOC DMRSF and shear wall requirements (see Tables 2 and 3). The changes in 1967 permitted some of the main beam reinforcement to be anchored outside of columns which resist a small percentage of the story-bent shear. Where confinement through a beam-column joint was required, ACI 318, Section 917(a) 3 was assumed to apply.

The 1968 SEAOC revision required:

1. Reinforced concrete shear walls and braced frames for all buildings to be in conformance to the SEAOC code
2. All structural elements below the base required to transmit seismic lateral forces to the foundation to conform to the SEAOC code
3. STR for full height of columns supporting shear walls or other rigid elements
4. Equal confinement for all combinations of concrete and steel
5. STR for a column whose ultimate capacity was less than the ultimate capacity shear of all beams framing into the column above the level being considered

The 1970 SEAOC revision [37] applied to RC shear walls. The variation in allowable shear stress with height to depth ratio was deleted because a careful re-evaluation of applicable test data did not fully support the prior provisions. The ultimate shear stress was related to the capacity reduction factor, concrete strength and reinforcement yield strength with a

TABLE 2

REQUIREMENTS FOR RC DMRSF

(Additional to ACI 318)

Provisions	1966 SEAOC	1971 SEAOC	1973 and 1974 SEAOC
General	Buildings > 160 ft have 25 percent DMSRF Buildings with K = 0.67 or 0.80 have DMRSF	Adds all concrete frames part of LFRS shall be DMRSF	Same
Design Method	Ultimate strength preferred. WS okay with equivalent FS	Same	Same
Concrete Strength	3000 psi minimum	Same, except 4000 psi maximum for lightweight	Same
Load Combination	U = 1.4 (D+L+E) U = 0.9D + 1.25E	U = 1.4 (D+L+E) U = 0.9D + 1.4E	
Reinforcing a. Material	ASTM A-15, A-408 (Structural or intermediate), or A-432	ASTM A-615, grade 40 or 60	Same
b. Yield	$f_y \leq 40$ ksi (flexural) $f_y \leq 60$ ksi (columns)	Actual yield $\leq$ fy + 18 ksi Ultimate Strength $\geq 1.33 f_y$	
Flexural Members a. Width-depth	Ratio > 0.4	Ratio > 0.3	Same

Table 2 (continued)

Provisions	1966 SEAOC	1971 SEAOC	1973 and 1974 SEAOC
b. Width	Ten in. or column width plus 0.75 beam depth each side of column	Same	Same
c. Reinforcing	Reinforcing ratio $p \leq 200/f_y$ both top and bottom. Two bars top and bottom full length. $p \leq 0.025$ nor $0.46\ f'_c\ p'/f_{y}p$ (negative M at column face) and positive M capacity $\leq 0.5\ M_{neg}$. Minimum of 0.25 or larger p' continue through beam	Same except Delete 0.46 f'p'/f p	Same Same
d. Splices	No tensile laps in tension areas unless confined. None in column or 2d from column. Two stirrup-ties minimum at each splice.	Same	Same
e. Anchorage	At exterior column, top and bottom reinforcing extends through column and 90° hook. Anchor in confined column core.	Same	Same
f. Web Reinforcing	Chapter 17, ACI-318 except: $V_u \geq \frac{M_u^A + M_u^B}{L} + 1.4V_{D+L}$	Same	Same
Stirrup Spacing	d/2 maximum throughout length	Same	Same

Table 2 (continued)

Provisions	1966 SEAOC	1971 SEAOC	1973 and 1974 SEAOC
Stirrup-ties	$s \leq d/4$, 16 bar diameter, or 12 in. Each end of member for 2d from support, at locations of possible inelastic deformation, or where p' required.	Same except $S \leq d/4$, 8 bar diameters. Twenty-four stirrup-tie diameter or 12 in. Provide lateral support for all longitudinal steel.	Same
Columns			
a. Shape	Ratio of maximum to minimum thickness ≤ 0.4, no dimension < 12 in.	Same	Same
b. Reinforcing	Tied columns, $0.01 \leq p \leq 0.06$	Same	Same
c. Splices	Center half of column. Lap 30 diameter or 16 in. minimum	Same except delete 16 in.	Same
d. Special Transverse Reinforcing	Through joint plus maximum column dimension, one-sixth clear height, but not less than 18 in.	Same, but add columns supporting shear walls or other rigid elements, special transverse reinforcing full height.	Same
Exception:	One-half of the above when beams on four sides		
Spirals	ACI-318, Section 913	Volume ratio $\geq$ ACI nor $p'' = 0.12\ f'_c/f''_{yh}$	Same
Hoops	Volume of hoops $\geq$ 2x required spirals	$A''_{sh} = 0.30\ ah'' \frac{f'_c}{f''_{sh}} (A_g - 1)$* nor $= 0.12\ ah'' \frac{f'_c}{f''}$	

Provisions	1966 SEAOC	1971 SEAOC	1973 and 1974 SEAOC
e. Effective Column Length	ACI 318, Section 915(d)	Same	Same
f. Beam-Column Joint Analysis	V_u = column shear plus beam shear with rebar stressed to f_y. Ultimate M capacity of column > than ultimate M capacity of beams framing into it.	Same	Same
g. Column Shear Transverse Reinforcing	$A_v f_y \frac{d}{s} = V_u - V_c$ If $P/A_g < 0.12 f'_c$, $V_c = 0$	Same	$A_v f_y \frac{dc}{s} = \frac{V_u}{\phi} - V_c$ (1974) ($V_c = 0$ when $\frac{P}{A_g} < 0.12 f'_c$)
Inspection	Special continuous inspection	Same	Same

* In 1967 Revision 0.03 was 0.45

TABLE 3

REQUIREMENTS FOR RC SHEAR WALLS AND BRACED FRAMES

(Additional to ACI 318)

Provisions	1966 SEAOC	1971 SEAOC	1973 and 1974 SEAOC
Design Method	Ultimate strength preferred, WS okay with equivalent FS	Same	Same
Load Combination	$U = 1.4\ (D+L+E)$ $U = 0.9D + 1.25E$ Use 2U for all but 100 percent MRSF	$U = 1.4\ (D+L) + 1.4E$ $U = 0.9D + 1.4E$ Use 2.8E for all but 100 percent MRSF	Same 1973 -- Use 2.8E for all but K = 0.67 1974 -- Use 2.0E for all but K = 0.67
Braced Frames	In DMRSF, special transverse reinforcing full length of bracing member	Same	Required for all buildings
Shear and Diagonal Tension	$v_u = \frac{V_u}{bd}$ $v_u \leq (0.8+4.6\ \frac{H}{D})\phi\sqrt{f'_c}$ $v_u \leq 10\phi\sqrt{f'}$ for H/D > 2 $\leq 5.4\phi\sqrt{f'_c}$ for H/D < 1	(1970 revisions) $v_u = V_u/A_c$ $v_u \leq 2\phi\sqrt{f'_c} + \phi\, p\, f_y$ (same p both directions)	$v_u = \frac{V_u}{\phi A_c}$ (1974) $v_{ult.} \leq 2\sqrt{f'_c} + p\, f_y$ (1974)

Table 3 (continued)

Provisions	1966 SEAOC	1971 SEAOC	1973 and 1974 SEAOC
Shear and Diagonal Tension (continued)	$V_u = v_c \, bd + V'_u$ $A_v = \frac{V'_u s}{\phi f_y d} (\frac{H}{D} - 1)$	v_u average $\leq 8\phi\sqrt{f'_c}$ v_u maximum $\leq 10\phi\sqrt{f'_c}$	
Vertical Boundary Members	For K = 0.80 buildings. Structural steel or confined RC column members. Anchor horizontal reinforcing in boundary elements.		
Reinforcing	ACI 318	$p = 0.0025$ each way $s = d/3$ or 18-in. special anchorage.	

maximum of 10 $\phi \sqrt{f'_c}$. Equal amounts of wall reinforcing were required in each direction with a minimum ratio, p, of 0.0025 each way. Wall reinforcement that resists shear was required to be anchored to wall boundary elements.

The 1971 SEAOC revision [37] made a number of major revisions. All RC space frames required to be part of the lateral force-resisting system (LFRS) were required to be DMRSF. All framing elements not required to be part of the LFRS were to be investigated for adequacy for vertical load carrying capacity at four times the distortion resulting from the code seismic forces.

The requirements for RC DMRSF were modified as follows:

1. Precast concrete was permitted providing the resulting construction complied with the SEAOC provisions.

2. The load factor, U, was increased from (0.90D+1.25E) to (0.90D+1.40E).

3. Lightweight concrete strength was limited to 4,000 psi.

4. Grade 60 reinforcing steel was permitted for beams with limitations.

5. A specific provision was added requiring all space frame members to be designed so they would not fail in shear if the frame is deformed beyond yield.

6. The excess of actual yield strength of reinforcement over the minimum specified must be taken into account in the design of required shear capacity of beams and columns. Ultimate shear capacities were to be computed with the ϕ factor reduction.

7. The minimum width-depth ratio for beams was reduced to 0.3 from 0.4.

8. The formulas for computing beam longitudinal steel were simplified.

9. The maximum stirrup spacing was reduced from 16 bar diameters to 8 bar diameters or 24 stirrup-tie diameters.

10. The calculation of confinement reinforcing was revised and confinement for columns 24 inches and less was reduced.

The Commentary presented sketches of the requirements for STR (see Figure 8), stirrup-ties (see Figure 9), and anchorage of beam reinforcement in column core (see Figure 10).

Two changes were made for shear walls: the load factor U was revised to conform to that for DMRSF, and 2.8E was to be used in calculating U for

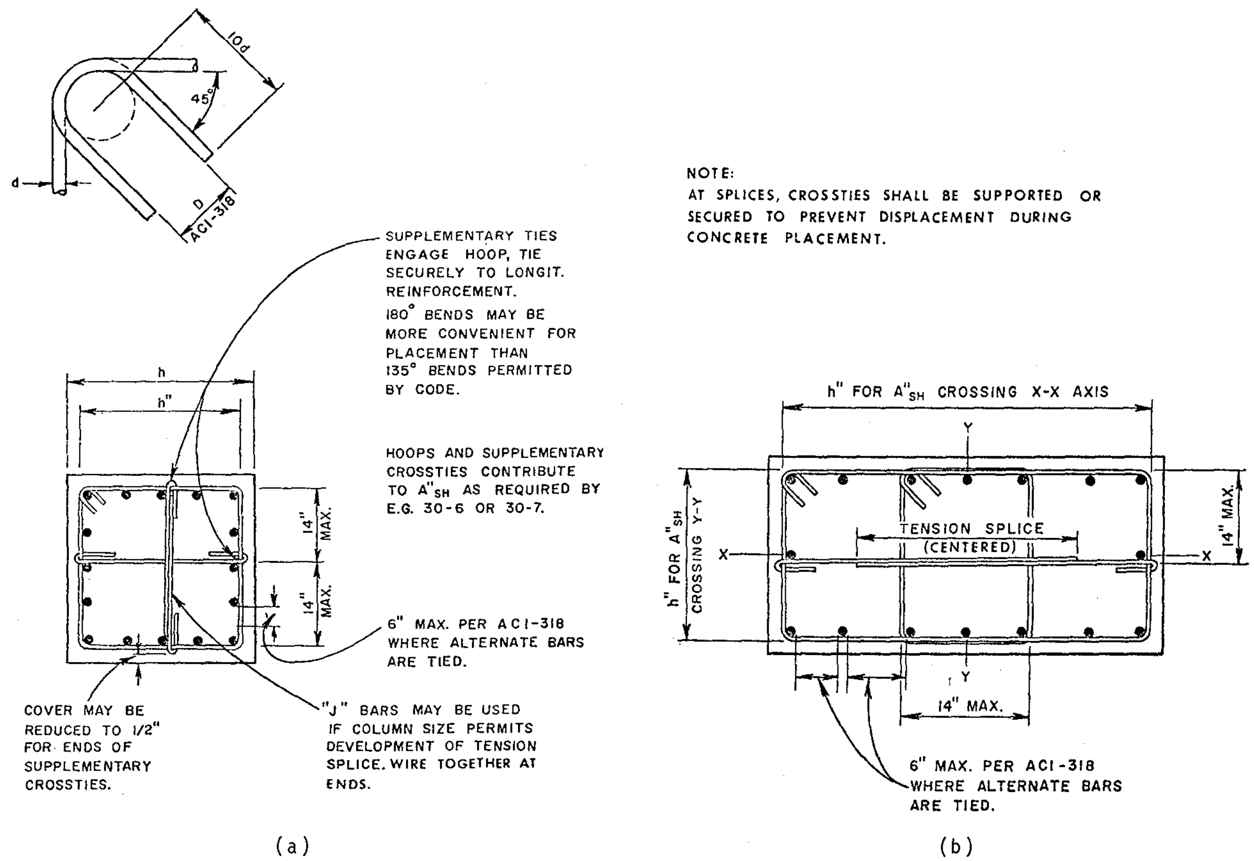


FIG. 8 STR REQUIREMENTS (from SEAOC BLUEBOOK, Ref. 37)

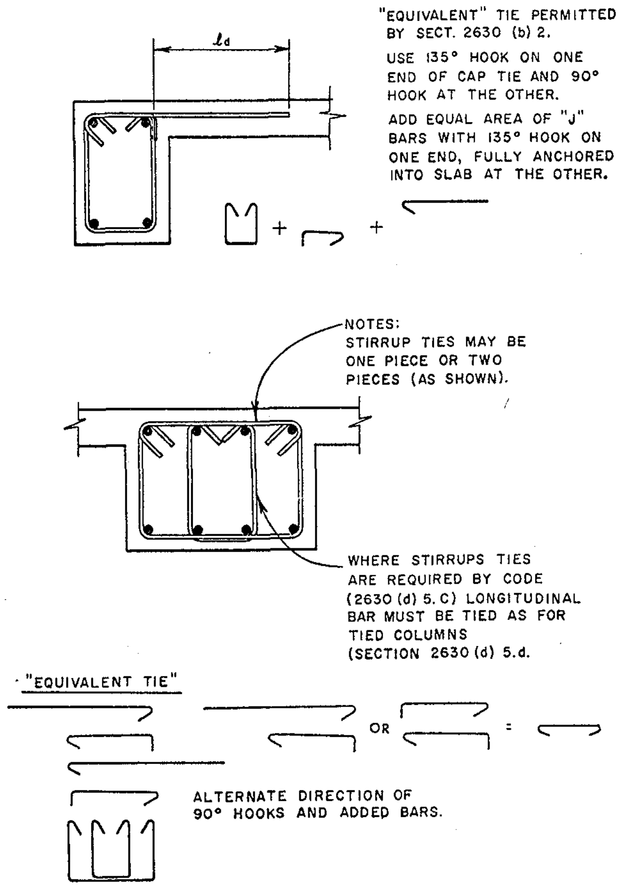


FIG. 9 STIRRUP TIES (from SEAOC BLUEBOOK, Ref. 37)

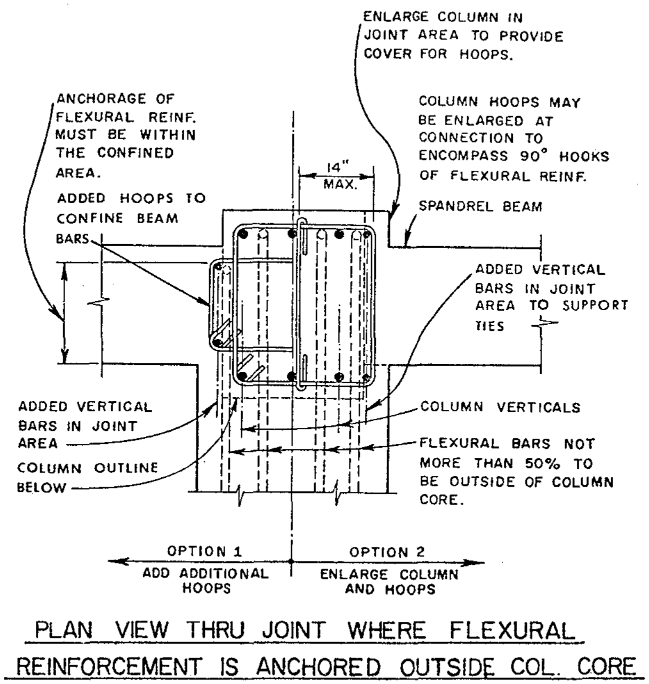


FIG. 10 ANCHORAGE OF BEAM REINFORCEMENT IN COLUMN CORE (from SEAOC BLUEBOOK, Ref. 37)

shear in lieu of the prior requirement of using 2U. This change was made to eliminate unreasonable shear requirements when E is a small part of the total U.

The ACI 318, 1971 edition [38] recognized the need for special seismic requirements for RC buildings in high seismic risk areas. An Appendix A was included that generally followed the SEAOC requirements. The major differences with the SEAOC provisions were the longer anchorage required by ACI for flexural member reinforcing in confined regions at beam-column joints, the larger volume of hoops required by ACI to obtain STR confinement, and the difference in the load factor U. ACT requires U = 1.025D + 1.27L + 1.47E, while SEAOC required U = 1.40(D+L+E).

The 1973 SEAOC revision [37] introduced a requirement for considering the structure's dynamic characteristics when distributing lateral forces in structures having irregular shapes or framing systems. DMRSF were required for all RC frames at the perimeter of all buildings except those with 100 percent shear walls. The concrete cover outside of the column core was deleted for purposes of calculating shear capacity of the concrete, V_c. STR was required for the full length of RC members of all braced frames. In addition, braced frame members were required to be designed for 1.5 times the forces calculated using the code-specified lateral forces. All of these changes were made to increase the ductility of RC structures and to minimize the possibilities of brittle-type failures. Recent earthquake damage experience made such changes desirable.

The 1974 SEAOC revision [37] was largely editorial and format except for shear walls and braced frames. For K = 0.67 and 0.80 buildings, the special ductility requirements were made to apply to all elements below the base which are required to transmit the seismic forces to the foundation. The load factor for calculating shear and diagonal tension in other than K = 0.67 buildings was changed from 2.8E to 2.0E. The Commentary was re-organized and expanded in some areas.

The ATC-3 Draft Provisions [39] were developed with the objective of being in a format suitable for adoption by jurisdictions in all areas of the United States. The ATC-3 provisions embody several concepts that are significant departures from present codes, such as:

1. More realistic ground motion intensities

2. Consideration of distant earthquake effects on long period buildings

3. Response reduction factors which are based on consideration of the inherent capacity for energy absorption, damping associated with inelastic response, and observed performance of various types of framing systems

4. Building design categories with variation of design and analysis requirements dependent upon seismic intensity

In addition, various methods for determining building response coefficients (periods) were evaluated and formulas recommended, and design material stresses approaching yield together with detailed design requirements for various materials are presented.

The total seismic force on a building is in the form V = CW, where W is the dead load of the building plus applicable other loads, and

$$C = \frac{1.2\ AS}{RT^{2/3}}$$

A is the appropriate acceleration coefficient, S is a site soil coefficient (varies from 1 to 1.5), R is the response reduction factor, and T is the structural response coefficient used to define the base shear and is related to fundamental period of the building. The R value is determined from Table 4 and is largely determined by the type of structural system employed. For example, one R value is given for ductile structural frames. The effect of construction material used is covered by specific material design requirements. The C_d values are used in determining the total story drift.

For purposes of seismic design, four building design categories (A through D) are as given in Table 5. The Seismic Hazard Exposure (SHE) Groups relate to building occupancy or use, with Group I being essential facilities, II is buildings with high density of occupancy, and III is all others. The Seismic Hazard Index is similar to seismic zoning in present codes.

The structural components in Category A buildings are required to be tied together, but no overall seismic design is required. In addition, Category B buildings need minimum seismic design such as collector elements, diaphragm design, design of bearing walls for seismic forces normal to the flat surface of the wall, reinforcement of openings in shear walls or diaphragms, and certain minimum pile foundation design requirements.

The design details of Category C buildings generally conform to the 1973 SEAOC provisions, while the design of Category D buildings is similar to the State of California Administrative Code requirements for hospitals.

The ATC-3 provisions for ERCBC construction basically follow the ACI 318-71 Appendix A with additions or modifications in several areas. It is understood that the proposed revisions of ACI Appendix A are in accord with ATC-3. The format is arranged to specify differing requirements for Category A, B, C, and D buildings.

In addition to the general requirements noted previously for Category A design, anchor bolts shall be enclosed by at least two ties. Allowable loads for anchor bolts will be given. The capacity reduction factors for shear are reduced from those in ACI 318-71. The factor for axial compression or axial compression combined with bending when axial stress due to all loads exceeds $0.1f'_c$ and the axial seismic stress exceeds $0.05f'_c$ and STR is not provided is 0.5.

TABLE 4

RESPONSE MODIFICATION COEFFICIENTS

FROM ATC-3-05 REPORT, JANUARY 1977

Structural System	Vertical Seismic Resisting System	Coefficients R	Coefficients C_d
BEARING WALL SYSTEM: A structural system without an essentially complete vertical load carrying space frame. Seismic force resistance is provided by shear walls or vertical trusses	Light framed walls with diaphragm sheathing	6	4
	Reinforced concrete or masonry shear walls	4	3
	Vertical bracing trusses	4	3.5
	Unreinforced masonry	1.2	1.2
BUILDING FRAME SYSTEMS: A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by shear walls or vertical trusses.	Light framed walls with diaphragm sheathing	6	4
	Reinforced concrete or reinforced masonry shear walls	5	3.5
	Vertical bracing trusses	4.5	3.5
	Unreinforced masonry	1.5	1.5
MOMENT-RESISTING SPACE FRAME SYSTEM: A structural system with essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by a moment-resisting space frame designed for the total prescribed seismic forces.	Ductile structural frame	7	5
	Structural steel frame	3.5	2.5
	Reinforced concrete frame	2	2
DUAL SYSTEM: A structural system with an essentially complete space frame providing support for vertical loads. A ductile moment-resisting space frame capable of resisting at least 25 percent of the prescribed seismic forces shall be provided. The total seismic resistance is provided by the combination of the ductile moment-resisting frame and shear walls or vertical trusses in proportion to their relative rigidities.	Reinforced concrete or reinforced masonry shear walls	6	4.5
	Wood sheathed shear panel	7	4
	Vertical bracing trusses	5	4
SPECIAL STRUCTURES: Inverted pendulum structures with the framing resisting the total prescribed seismic forces and providing support for vertical load.	Ductile frame	4	3
	Structural steel frame	2	2

TABLE 5

BUILDING DESIGN CATEGORY

FROM ATC-3-05 REPORT, JANUARY 1977

Seismic Hazard Index	Seismic Hazard Exposure Group		
	I	II	III
1	B	A	A
2	B	B	B
3	C	C	B
4	D	C	C

ERCBC buildings in Category B are required to meet many of the requirements of Appendix A for flexural members such as limits on tensile reinforcing, continuous reinforcing top and bottom, anchorage, and web reinforcement. Members subjected to bending and axial load having a design axial compressive force exceeding $0.1f'_c\ A_g$ shall have lateral reinforcement continued through the joint.

Category C and D ERCBC buildings are required to meet 1974 SEAOC DMRSF provisions. In addition, the axial compressive force in a flexural member is limited to $0.10f'_c\ A_g$ and the clear span shall not be less than four times its effective depth. Elements containing lightweight aggregate concrete shall have 1.25 times the STR required for normal weight concrete. The volumetric ratio of STR shall be based on the volume of concrete bounded by lines joining the centers of the peripheral column bars. The yield strength of the STR shall be not less than the yield strength of the longitudinal reinforcement.

There are several requirements for RC shear walls and braced frames in addition to those in the 1974 SEAOC provisions. Two curtains of reinforcement are required in shear walls when the unit shear is $\geq 2\sqrt{f'_c}$. The allowable shear strength for lightweight concrete shear walls is 0.75 of that for walls of normal weight concrete. Shear walls in all buildings except those with DMRSF where the maximum combined design compressive stresses exceed $0.2f'_c$ shall have vertical boundary elements as specified in the 1974 SEAOC. These boundary elements may be discontinued at a level where the calculated stress is less than $0.15f'_c$. A similar requirement is specified for reinforced concrete diaphragms and for openings in shear walls and diaphragms. For braced frame members where the design stresses exceed $0.2f'_c$, STR shall be provided full length. Construction joints in shear walls and diaphragms shall be designed to resist the design forces at the joint. For joints dependent on dowel action and friction on a roughened concrete surface, the shear transfer capacity at the joint equals the capacity reduction factor times the sum of the shear strength of the dowels plus 0.75 times the net compressive design force. For lightweight concrete, the shear transfer capacity is 0.75 times that for normal weight concrete.

COUNTRIES OTHER THAN THE UNITED STATES

The design requirements for ERCBC in other countries often are not as detailed as those in SEAOC or ACI. Brief summaries of the requirements in several countries with seismic exposure follow.

New Zealand [40, 41] - All structures must have some ductility. Ductile frames and coupled shear wall structures have the lowest seismic design loads but stringent ductility design requirements. The required design procedure is called "capacity design" in which energy dissipating elements or mechanisms are selected and then properly designed and detailed. Other elements are then designed with reserve strength capacity to ensure that the structure responds as intended in the design. It appears that detail requirements similar to SEAOC or ACI requirements for ERCBC are followed.

Portugal [42] - Seismic coefficients for simple structures are determined taking into account the seismicity, soil characteristics, and dynamic properties and ductility of the structure. Complex and/or important structures are designed using seismic motion response spectra and dynamic linear and nonlinear analyses as appropriate. Structures are checked for safety at ultimate limit states. Specified ductility factors are 3.0 for high-ductility RC frames, 2.0 for shear wall-frame structures, and 1.5 for shear wall structures.

Columbia [43] - A proposed seismic code requires that the fundamental period of the structure, T, be greater than 1.4 T' or less than 0.7T', where T' is the period of the site. This provision is intended to minimize the seismic forces transmitted to a building by avoiding resonance between the soil and the building. Buildings over 60 meters in height require a rigorous dynamic analysis. Lower structures are to be proportioned for elastic behavior for the normal design earthquake and have enough ductility to avoid collapse for a maximum earthquake. "Ductility ratio" is defined as the ratio of the curvature at ultimate and yield states of members. Tall structures (> 60 m) must have a minimum ductility of 6 and lower structures must have a minimum of 4. The requirements for RC construction appear to be similar to the SEAOC and ACI requirements.

Australia [44] - The latest draft of the Australian code has requirements analogous to those in the 1973 SEAOC for ERCBC in the highest seismic zone. The requirements for RC DMRSF, shear walls, and braced frame members appear to be identical to those in the 1973 SEAOC.

PROBLEMS IN INTRODUCING RESEARCH RESULTS AND IMPLEMENTING THEM IN PRACTICE

There appear to be several problems generally encountered when introducing research results into codes and later implementing them in practice. One major problem is to duplicate in laboratory testing the situation usually encountered in actual buildings. Size of the test model is often a limitation, although laboratory facilities in the past few years have been expanded and funding provided for larger-scale testing. The full-scale testing is usually limited to one beam span plus beam and column stubs or similar type arrangements. Testing can be done with cyclic-reversible loading but facilities are not available to perform tests on full-scale models using vibratory loading.

Another problem associated with full-scale testing is that of developing test specimens representative of the range of loading conditions, configurations, and sizes encountered in building design.

The results from scale model testing indicate the type of solution or requirement needed for some design situations. However, actual conditions, scaling factors, and variations in construction often make it difficult if not impossible to develop reasonable criteria; therefore, a conservative solution is selected.

A problem often arises with implementing ERCBC research results because the recommendations involve complex details both in design and construction. Research has shown the need for confinement of the concrete and anchorage of flexural reinforcement. Figures 11(a) and (b) show typical interior and exterior beam-column joints and illustrate the complexity and congestion in joints. The sizing of the beams and columns are often controlled by the need to continue longitudinal beam steel through the joint and at the same time miss the vertical column bars. Then the confinement hoops have to be placed and enough room left for concrete placement.

The detailing of the joints to ensure viability of construction requires considerable design time. Unless the designer is familiar with all of the design detail requirements, the possible congestion at a joint, and the consequent effect on member sizes, he may find that he has to resize members (and advise the architect of the larger sizes required). The result can be considerable extra work.

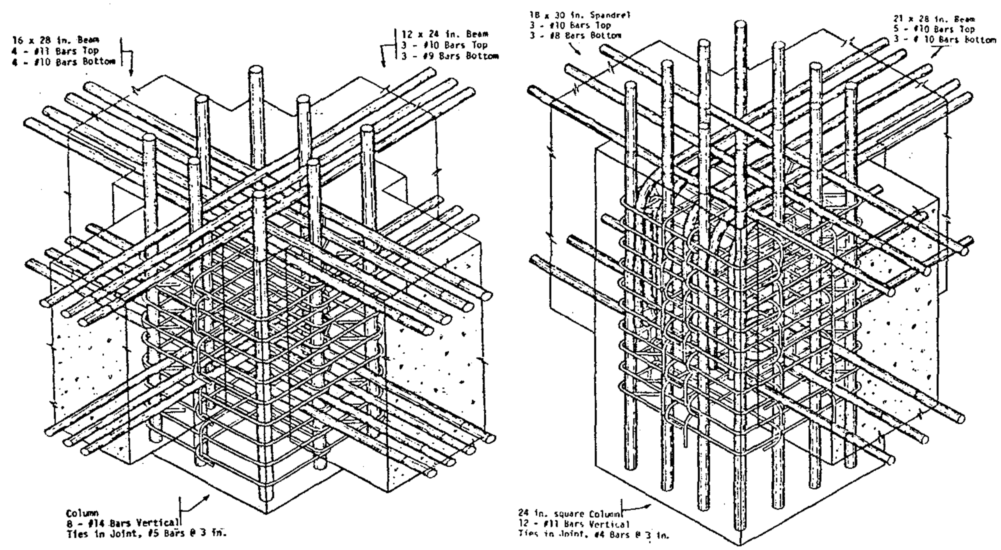
Many of the research results, when put in equation form for presentation in ERCBC code provisions, appear to be complex and therefore tend to give the implication that the design of RC and ERCBC is a precise technique. This complexity and appearance of rigorous accuracy tend to increase resistance to their acceptance in practice because often the practicing professional does not understand the development of the equations and therefore is hesitant to accept and use them. The experienced engineer realizes that the input data contains numerous assumptions and hence extreme accuracy in calculation is not always appropriate. The complexity often increases the time required for designs.

The Commentary published with the SEAOC provisions is helpful to the designer of ERCBC. The Commentary explains the intent and reasons for the requirements. The ACI-ASCE Committee 352 report [45] presents recommendations and design examples for beam-column joints where the column is equal to or wider than the beam width. The Committee's recommendations were based on laboratory and field experience and reflect their evaluation of much of the RC beam-joint research performed from 1971 through 1975.

CONCLUSIONS AND RECOMMENDATIONS

Development of code requirements for ERCBC has shown considerable progress in the past two decades. Observations of earthquake damage to RC buildings and evaluations of the results of research have led to development of most of the code provisions. There are many areas that need further research and study.

1. Because of the rising costs of construction and the continuing search for cost reduction, there is increasing pressure to develop details for precast frame members that will provide the required ductility. The connections of floor and roof slabs to the frame members also need study.



Design example No. 3—Interior joint, Type 2

Design example No. 2—Exterior joint, Type 2

(a)

(b)

FIG. 11 BEAM-COLUMN JOINT DETAILS (from ACI-ASCE COMMITTEE 352 REPORT, REF. 45)

2. The development and use of details for prestressed and post-stressed RC construction necessary to provide ductility need extensive study. The ability of such structural systems to develop the required strength and ductility should be further investigated.

3. Further research is needed to develop requirements for flat slab-column or flat slab-wall systems so as to provide adequate ductility in the inelastic range.

4. Guidelines for the calculation of fundamental period of RC frames and shear wall buildings for use in analysis and design are needed. Should cracked or uncracked sections, clear span dimensions, gross section dimensions, etc., be used. For example, after study of available data, the ATC-3 committee recommended the same simplified formula that has been used for many years to calculate the period of RC buildings other than frames ($T_R = 0.05\ h_n/\sqrt{D}$).

5. Coupled shear walls are often used in buildings. The question of whether the coupling beams should be designed and detailed to provide the required shear and moment strength and ductility should be evaluated. The use of diagonal principal reinforcement in the coupling beams improves their strength and ductility, but is expensive to construct.

6. Considerable research is still needed in beam-column joint design. ACI-ASCE Committee 352 published a lengthy list of needed research (see Appendix A hereto).

7. The inelastic behavior of shear wall-frame systems needs further study [46].

It is hoped that wherever possible research results can be presented in somewhat simplified format so as to make them more readily acceptable and usable in practice.

It is recommended that the improvement of communications between the researcher and the practicing engineer be given detailed study. Some research projects utilize advisory panels -- others do not. The advisory panel concept is helpful for some projects and for others it is ineffective. One possible solution might be to make a conscious effort to involve practicing professionals as consultants who would be involved in the planning stages and as the project progresses. The consultants could assist in presenting the research results in a format compatible with use in codes and by practicing engineers. Obviously such participation would take a considerable amount of the consultants' time. Some means should be developed for the consultants to be reimbursed for their time and expenses because most structural engineering firms are relatively small and the amount of time required would be a burden if it were donated.

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APPENDIX A—AREAS OF NEEDED RESEARCH

(from ACI JOURNAL, Ref. 45)

In developing "Recommendations for Design of Beam-Column Joints in Monolithic Concrete Structures," the committee surveyed available research into the problems of joint behavior and, wherever possible, incorporated the research results into the recommendations. However, it is apparent that a number of problems have not been studied sufficiently to warrant making specific recommendations and in some cases there is no research reported in the literature which bears on the problem. To identify the areas of needed research and to provide an indication of some of the limitations of the recommendations, the following listing of topics was developed.

A.1—Effectiveness of confinement due to spiral reinforcement or rectangular ties and crossties

The behavior of confined concrete under different combined states of stress that can be developed in a joint should be investigated. Most of the data available are from tests under uniform compressive stresses.

A.2—Influence of lateral members framing into the joint

Very limited test data indicate that beams framing into the core have a beneficial effect on the shear strength of the core. However, the confining influence of such members has not been studied systematically and considerable work needs to be done to evaluate the factor γ included in the recommendations. Confinement and forces produced by lateral beams and floor slabs should be considered in such tests.

A.3—Effective core area for shear calculations

Tabulations of the shear strength of the concrete in core are dependent on the effective core area selected. A key question is the manner in which concrete cover over the transverse reinforcement should be treated. Studies are needed to evaluate the effectiveness of concrete cover for shear strength in the presence or absence of lateral members framing into the joint.

A.4—Influence of biaxial forces on shear strength

All beam-column joint studies reported are limited to the application of loads or deformations producing shear in one of the principal directions of the joint. Work is needed to evaluate the influence of biaxial forces acting on the core. It is important that beams in both directions are sub jected to seismic loadings so that the influence c biaxial shear (and torsional forces) on the core a well as damage to "confining" beams be fully ex plored. Studies are also needed to evaluate th behavior of joints with beams and columns nc arranged concentrically on the joint core so th torsional stresses are induced.

A.5—Influence of axial column loads

The equation presented in the recommendatior for shear strength of the core concrete indicat that compressive axial loads are beneficial. In th absence of test data, the shear strength is assume to be zero where known tensile forces act on th column. For Type 2 joints, the tensile forces ca be induced by vertical ground motions and/or th overturning moments due to lateral excitatic which are highly variable.

At present there are no data available regardir the shear and flexural behavior of columns und such excitations.

A.6—Shear strength of joint

The recommendations include a provision limi ing ultimate shear stress on the core to $20\sqrt{f_c'}$ ar the contribution of transverse reinforcement $15\sqrt{f_c'}$. These values must be examined in light tests in which the influence of biaxial forc lateral members framing into the joint, and cc umn tensile forces are evaluated.

A.7—Size and location of members framing in joint

The recommendations are limited to cases whe the beams are no wider than the columns a beam reinforcement is located within the colun reinforcement. Research is needed to evaluate t performance of the joint where the beam is wid than the column and all the beam reinforceme may not be placed within the column bars. Whe beams are not concentric with the column, torsi may be produced in the joint and add to the she stresses. No data are available on the effect of (centric beam locations.

A.8—Column hinging

To prevent shear strength deterioration in t columns, the recommendations call for joint tra verse reinforcement to continue into the colum for a distance equal to the effective depth of t column. In addition, the flexural capacity of t columns at the joint must not be less than that of the beams framing into the joint. The provisions are intended to insure that hinges form only in the flexural members. Studies are needed to evaluate the consequences of column hinging on performance of structures.

A.9—Anchorage capacity of hooked bars

The recommendations include provisions for calculating the capacity of standard hooks under certain conditions of confinement. Tests are needed to evaluate the confining influence of members framing into the joint normal to the plane of the hooked bar. Such tests should include an evaluation of bars passing through the joint in the vicinity of the inside radius of the hooks and should also be conducted with loads on the lateral beams. Additional straight lead embedments are calculated using the development length equation of ACI 318-71 modified to reflect an assumed beneficial influence of lateral confinement. The basis for calculating straight lead embedments remains to be established.

A.10—Anchorage of straight bars through joints

Under racking loads straight bars which extend through a joint may be subjected to a tensile force at one critical section and to a compressive force at the opposite face. This combination of forces coupled with severe load reversals may lead to a rapid deterioration of the anchorage capacity of the bar. Studies are needed to evaluate the magnitude of the problem and to develop means of improving anchorage capacity.

A.11—Lightweight aggregate concrete

Studies are needed to evaluate all aspects of joint behavior where lightweight aggregate concrete is used. Tests are needed to evaluate the ductility of confined sections under axial load, shear strength of joint cores and members under load reversal, and anchorage capacities of both straight and hooked bars.

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
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SUMMARY OF PRESENT CODES AND STANDARDS IN THE WORLD

by

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INTRODUCTION

The purpose of this paper is the introduction of basic ideas of earthquake resistant regulations in the world except U.S.A. A recent Draft of Earthquake Resistant Regulation in Japan is explained especially in detail. The basic philosophy of earthquake resistant design of building structures are 1) to prevent loss of human life and personal injury, 2) to minimize damage to property, and 3) to ensure vital services, in the event of earthquakes.

It is a well recognized fact, however, that to provide complete protection against all earthquakes is not economically feasible. It is generally accepted that earthquake design forces should be estimated in order 1) to prevent structural damage and minimize other damage in moderate earthquakes which occasionally occur, and 2) to avoid collapse or serious damage in severe earthquakes which very seldom occur.

General Procedure

The seismic analysis of a structure is usually perfomed by a method using equivalent static loadings to represent the dynamic actions of the earthquake upon the structure. In this case the lateral seismic forces to be distributed over the height of the building are determined according to one of the following methods:

1) The total seismic force on a structure is determined by the so-called base shear coefficient. For a further step, the total seismic force is distributed over the height of the structure, by considering the response of the structure during earthquakes.

2) The lateral seismic force at each floor level is directly determined by so-called lateral seismic coefficients. These coefficients are generally varied over the height of the building, in consideration of the response of the structure during earthquakes.

Either of these methods should give a similar result when applied properly.

The value of the base shear coefficient or the lateral seismic coefficient is usually evaluated by considering the following factors:

1) Dynamic properties of the structure (natural periods of vibration, modal shapes, damping)

2) Type of construction (ductility or energy-absorptive capacity of the structure)

3) Importance of the structure as related to its use

4) Seismicity of the region

5) Subsoil Conditions

6) Allowable stresses and load factors

The force induced by an earthquake on a structure may act in any direction. However, only horizontal components of the earthquake forces are usually considered and these are generally taken to act nonconcurrently along the two main axes of the structure, although some consideration is now being given to their concurrent action.

Vertical seismic forces also should be considered in the design of a structure and/or portions of the structure when it is deemed necessary.

The value of vertical load to be used for seismic calculations is equal to the total dead loads plus a probable live load for the structure under consideration. In heavy snow areas, the probable snow load is also considered.

Larger seismic coefficients than those for the building as a whole are usually applied to the design of parts or portions of buildings such as cantilever parapets, supported structures (towers, tanks, penthouses, chimneys, etc.) projecting from the roof, ornamentations, appendages, etc.

The provisions prescribed in seismic codes in connection with earthquake loading and response criteria include 1) distribution of horizontal shear, 2) evaluation of overturning moments and horizontal torsional moments, 3) drift limitation, 4) separation of buildings, 5) setbacks, 6) structural design requirements including the problems of providing the necessary ductility, etc. These are the essential items to achieve sound structural design of buildings to resist earthquakes.

In some earthquake countries, the earthquake design forces are determined by a dynamic analysis due to design earthquake motions selected to be appropriate for site and soil conditions. This method is recommended and may be required for specific structures such as slender high-rise buildings and especially important structures involving unusual risks. The elastic and inelastic dynamic analyses based on time-history accelerograms are used for the aseismic design of tall buildings in some countries.

CURRENT SEISMIC CODE REQUIREMENTS

Horizontal Earthquake Force and its Distribution

The earthquake loading prescribed in most current seismic codes in the world may be represented as follows:

$$F = C(Z,I,S,K,T)W = \Sigma f_i$$

or

$$f_i = k_i(Z,I,S,K,)w_i$$

where
- F = total earthquake force or shear at the base of the structure.
- f_i = lateral seismic force applied to the level designated i.
- C = seismic base shear coefficient which is determined in consideration of Z, I, S, K and T.
- k_i = lateral seismic coefficient assigned to level i which is determined in consideration of Z, I, S, K and T.
- W = total vertical load used for seismic calculations.
- w_i = portion of W which is located at or is assigned to level i.
- Z = seismicity of the region (seismic zoning factor).
- I = importance of the structure as related to its use (importance factor).
- S = subsoil condition (soil factor)
- K = type of construction, damping, ductility or energy-absorptive capacity of the structure (construction factor).
- T = natural period of vibration of the structure in the direction under consideration.

The horizontal earthquake force and its distribution prescribed in some of the typical seismic codes are explained in the following chapter.

<u>CANADA</u> : National Buidling Code of Canada

$$F = C(Z,I,S,K,T)W = Z\cdot I\cdot S\cdot K\,\frac{0.05}{\sqrt[3]{T}}\,W$$

$Z = 1,\ 0.5,\ 0.25$
$I = 1.3,\ 1.0$
$S = 1.5,\ 1.0$
$K = 1.33,\ 1.0,\ 0.8,\ 0.67$.

$$f_i = \frac{(F-f_t)w_i h_i}{\sum_{x=1}^{n} w_x h_x} \qquad F = \sum_{i=1}^{n} f_i + f_t$$

$f_t = 0.07\ TW \leq 0.25W$
$f_t = 0$ for $T \leq 0.7$

where f_t = concentrated load at the top of the slender structure
T = fundamental period of vibration (sec.)
n = number of the maximum stories

The design earthquake loading may be determined by dynamic analysis.

<u>NEW ZEALAND</u> : New Zealand Standard Code of Practice for General Structural Design and Design Loadings (NZS4203), (1976)

$$F = C(Z,I,R,S,K,T)W = (Z,S,T)I\cdot R\cdot K\cdot W$$
$$= C_0\cdot I\cdot R\cdot K\cdot W$$

For rigid and intermediate soils,

$C_0 = 0.15,\ (0.125),\ [0.1]$ $\quad T \leq 0.45$ $\quad$ no parenthesis : Zone A
$C_0 = 0.075,\ (0.0625),\ [0.05]$ $\quad T \geq 1.2$ $\quad$ round blacket ( ) : Zone B
C_0 = linear decreasing with period T $\quad 0.45<T<1.2$ $\quad$ square blacket [ ] : Zone C

For flexible soil,

$C_0 = 0.15,\ (0.125),\ [0.1]$ $\quad T \leq 0.6,\ (0.7)\ [0.8]$

C_0 = 0.0825, (0.075), [0.065] $T \geq 1.2$
C_0 = linear change 0.6, (0.7), [0.8]<T<1.2

I = 1.0, 1.3, 1.6
R = risk factor = 1.0, 1.1, 2.0, 3.0
K = K' x M
K' = structural type factor = 0.8, 1.0, 1.2, 1.6, 2.0, 2.5
M = structural material factor = 0.8, 1.0, 1.2
$0.04 \leq C \leq 3.6\ C_0 \cdot I \cdot M$

$$f_i = \frac{(F-f_t)w_i h_i}{\sum_{x=1}^{n} w_x h_x}, \quad F = \sum_{x=1}^{n} f_i + f_t$$

$f_t = 0.1F$ for slender structures

Dynamic analysis using spectral modal analysis is allowed for any building and may be required for special structures. Lower limit of the base shear for dynamic analysis is 0.9F of static analysis. Time history response analysis may be used to supplement the spectral modal analysis.

VENEZUELA : Provisional Standard for Earthquake-Resistant Structures, December 26, 1976

$$F = C(Z,I,S,K)W = Z[I \cdot S \cdot K]W$$
Z = 1, 0.5, 0.25
$[I \cdot S \cdot K]$ = 0.045, 0.15

$$f_i = F \frac{w_i h_i}{\sum_{x=1}^{n} w_x h_x}$$

The above mentioned formulae are applied for buildings less that 20 floors or less than 60 meters in height. For designing buildings taller than the above, dynamic analysis is required in addition to the static design method. The final stresses to be used for design shall not be less than 60% of those due to the static design method.

U.S.S.R. : Standards and Regulations for Buildings in Seismic Regions (1970)

$$f_{ki} = k_{ki}(Z,I,T_k)w_i = [Z \cdot I] \cdot \beta_k \cdot \eta_{ki} \cdot w_i$$

where f_{ki} = design seismic force acting i in k th vibrational mode
T_k = natural period of k th mode
$[Z \cdot I]$ = 0.1, 0.05, 0.025

$\beta_k = \frac{1}{T_k}$, $0.8 \leq \beta_k \leq 3$, β_k : increased for very slender structure

$$\eta_{ki} = \frac{x_{ki} \sum_{j=1}^{n} w_j x_{kj}}{\sum_{j=1}^{n} w_j x_{kj}}$$

x_{kj}, x_{ki} = deflection at j and i in K the mode

In designing most structues, only the fundamental mode of vibration of the structure need be considered. For tower-like structures (height/width $\geq$ 5) and flexible frame structues (T_1 > 0.5 sec.), higher modes (up to the third mode) of vibration should be considered. In this case, the stresses (moments, axial and shear forces) of the structure are computed by the following formula:

$$N = \sqrt{N^2_{max} + 0.5\sum_{1}^{n} N_i{}^2}$$

where N = value under consideration

N_{max} = maximum value among all modes of vibration

N_i = other value except N_{max}

The seismic stress induced in buildings higher than 5 stories should be multiplied by the following factor:

$$\alpha = 1 + 0.1\ (n-5) \leq 1.5$$

n = number of the maximum stories

<u>INDIA</u> : Criteria for Earthquake Resistant Design of Structures, Jan. 1970 (IS: 1893 - 1970, Second Revision)

$$F = C(Z,I,S,I,T)W = \alpha_h \cdot I \cdot S \cdot K \frac{0.5}{\sqrt[3]{T}} W$$

α_h = 0.08, 0.05, 0.04

I = 1.0, 1.5

S = 1.0, 1.2, 1.5

K = 1.33 max. and 0.33 min.

$$f_i = \frac{F \cdot w_i h^2{}_i}{\sum_{i=1}^{n} w_i h^2{}_i}$$

The above mentioned formulae are applied for buildings lower than 40 m in height. For buildings taller than 40 m in height and up to 90m, modal analysis is recommended. For buildings taller that 90 m, detailed dynamic analysis shall be made based on expected ground motions.

Vertical Earthquake Force

Earthquake ground motions are in three dimensions, including both horizontal and vertical components. The vertical component is usually less intense than either horizontal component and is usually characterized by higher frequencies. In the vicinity of the epicenter, however vertical acceleration may be higher.

Recent records obtained by strong motion seismographs indicate that considerable values of vertical acceleration have been observed at the upper floors of high rise buildings.

In most seismic countries the vertical seismic forces due to earthquakes are not considered for the design of structure except for the effect of uplift forces and for very important structures such as reactor buildings in nuclear power stations. It should be noted, however, that the vertical seismic coefficients of 1.2 to 1.4 are usually considered for the seismic design of buildings in Italy and France.

Although no requirements are provided for concurrent vertical and horizontal forces in most seismic codes, the recent accelerograms obtained have suggested that effect due to veritcal component should be investigated.

Seismic Regionalization

Seismicity of the region for the construction site is usually indicated by a seismic zoning map, which may be determined either from the seismic history of the region or on the bases of seismotectonic factors, or from a combination of these approaches.

The maximum intensity of earthquakes to be expected in a region in a given future period of time, say about 100 years, is sometimes considered as the basis of the local seismicity.

The intensities of the earthquake motions are sometimes given in terms of the Modified Mercalli Intensity Scale (1931). The intensity of the most seismically severe zone is usually IX of the M.M. Scale which may correspond to 0.3g to 0.5g in ground acceleration.

Importance of The Structure As Related To its Use

Some seismic codes include classifications of strucutres depending upon what is called the importance of the structure. Larger static forces are frequently required for

1) buildings which have essential functions for the safety of public, after the hazardous earthquake

2) buildings where a large number of people assemble

3) buildings, the collapse of which endanger the surrounding public

The first category includes hospitals, emergency relief stores, fire stations, telephone exchanges, broadcasting and television buildings, power stations, etc. These structures should remain in operation after an earthquake. The second category includes assembly buildings, schools, theaters, etc. where many human lives in each structure may be endangered in case of collapse. The third category includes nuclear power plants and chemical plants.

The value of the importance factor employed in the seismic codes is usually in the range of 1.2 to 1.5 in the evaluation of earthquake forces. In U.S.S.R. and other seismic countries in eastern Europe much larger values (2 and 4) are used in the aseismic design of important structures.

Subsoil Conditions

In some seismic codes, the effects of subsoil conditions are taken into account independently or in combination with the type of construction of the building for evaluation the earthquake forces.

It is recognized that some components of the seismic ground motion are magnified on soft subsoil layer and that tall flexible buildings constructed on soft soil may suffer greater damage than those on hard layers.

On the other hand, it is generally recognized that the motion of the ground at a particular site during an earthquake has a characteristic period of vibration which tends to be short on firm ground and long on soft ground. Therefore, attention should be paid to the problem of the resonance of a structure with the ground motion, together with the complex interaction between them.

In the evaluation of earthquake forces, the ratio of soil factor for soft soil to that for hard soil is generally taken in the range of 1.5 to 2. In some countries such as Chile and Mexico, the soil factors are given in combination with the type of construction, taking their dynamic properties into consideration.

Types of Construction Related with Ductility

The overall ductile property of a structure provides an important contribution to its earthquake resistance. The capability of the structure to absorb a large amount of energy in the inelastic range is essential to avoid catastrophic failure.

It is recognized that moment-resisting frames of ductile materials such as structural steel and ductile reinforced concrete have shown good earthquake resistant characteristics. In the case of reinforced concrete buildings, however, some structures are subjected to the brittle shear failure. So provisions to avoid shear failures in such building should be followed.

In some seismic codes such as the U.S.A. and Canadian codes, the coefficient K is assigned to different types of structural systems. The ratio of the maximum value of K to the minimum is 2 as shown in paragraph 3.1.

It is interesting to note that the construction factors are considered in relation to the damping of the structure in the Rumanian seismic code.

Drift Limitations

Control of lateral deflection or drift of a story relative to its adjacent stories is considered to deal with the problems of 1) restriction of damage to the non-structural components such as glass panels, curtain wall panels, plaster walls and other partitions, 2) reduce panic or discomfort due to large motions and 3) control of building stability in inelastic deformations.

In should be noted that realistic interstory drift during earthquakes are best estimated by computations using a method of dynamic analysis. However, drift limitations using design static earthquake loading are given in some seismic codes. The values of the interstory drift are limited to 0.002 of the story height under the seismic design loadings in Mexico.

In Japan there is also no provision for drift limitation in the Building Standard Law, but the value of 2 cm per story is usually taken as the limit of drift for high rise buildings when computed by a method of dynamic analysis.

In connection with the problem of drift, there are provisions for the separation of buildings in some seismic codes such as U.S.S.R., Venezuela, Mexican and Portuguese Codes to avoid hummering reactions due to earthquakes.

Horizontal Torsional Moments

The torsional effects of earthquake forces are considered in the aseismic design of a structure with eccentric mass distribution.

Considering the inaccuracy of estimating eccentricity, the seismic codes of some countries such as Mexico, Canada and U.S.A. consider a larger eccentricity than the computed ones. When the eccentricity thus increased is relatively large in comparison with the corresponding plan dimension, the effect of torsion is doubled or a dynamic analysis is required in the case of the Canadian seismic code.

Overturning Moment

As a building (especially a tall building) is excited by the fundamental but also higher modes of vibration, the evaluation of the overturning moment at a given level of the building should be made by a method of dynamic analysis.

The distribution of equivalent static earthquake forces prescribed in the seismic codes primarily reflects the forces that may be developed by the dominant fundamental mode. Therefore it has been considered that the overturning moments computed by the distributed earthquake forces prescribed in the code may be conservative for the calculation of the axial loads from earthquake forces on vertical elements and footings.

In the seismic codes of Canada, the provisions on the reduction factors of the overturning moment at any given level are prescribed. The value of the reduction increases with the increase of the fundamental period of vibration of the building and also with the distance from the top of the building to the lower level under consideration.

Dynamic Analysis

The usual dynamic analysis procedure is to use either the earthquake response spectra or time-history accelerograms as the basis of design.

Modal analysis procedures based on earthquake response spectra are prescribed in the seismic codes of India, Mexico, New Zealand, Peru, Rumania, U.S.S.R. and Yugoslavia. In the modal analysis the maximum dynamic response is usually obtained by the method of "Square root of sum of squares, "taking the first three vibrational modes into consideration.

Dynamic analysis based on the time-history accelerograms of appropriate earthquakes are recommended for aseismic design of tall buildings in the seismic codes of India and Canada.

When the structural response obtained by the dynamic analysis is not satisfactory, the assumed structural model is modified and a revised response is computed. This procedure is repeated until a sound structural design of the building is accomplished. In tall buildings there may be many items of mechanical and electrical equipment and piping, some of which should ratain their functions during and after a severe earthquake. The aseismic design of these installations should be made by dynamic analysis procedures based on the earthquake response of the portions of the building which support them.

A DRAFT OF EARTHQUAKE RESISTANT REGULATIONS IN JAPAN

This Draft of Earthquake Resistant Regulations owes a great deal to the results of investigations of the major ministrial project on new technology for synthesized earthquake resistant design in Ministry of Construction, which had been carried out from 1972 through 1976. I also has reflected the results of investigations in universities, public and private laboratories. To establish a practical and rational earthquake resistant method is the purpose of the Draft.

Outline of Draft of Earthquake Resistant Regulations

Damages of structures caused by Niigata Earthquake (1964), Tokachioki Earthquake (1968) and San Fernando Earthquake (1971) stimulated to improve and rationalize the earthquake resistant regulations. For this purpose, the new major ministrial project and been carried out from 1972 through 1976 and A Proposal For Earthquake Resistant Regulations was presented by Ministry of Construction in March 1977. This project consisted of 6 themes and 20 sub-themes.

The proposal has been devided into 2 parts. In part I, a fundamental conception commonly applied to bridges, soil structures, underground structures and buildings is proposed. Part II deals with the calculation methods of buildings. The draft is compiled according to the following principles.

(1)Standardization of Fundamental Conception--Each earthquake resistant regulation has its own background. It has improved by itself by studying the damages of structures caused by past earthquakes, analyzing the earthquake ground motions which have been obtained by the strong motion observation network and investigating the characteristics of structures and their members. Consequently the design methods have been diversified in accordance with the uses and types of each structure.

In the project, existing earthquake resistant regulations were reviewed and the fundamental conception for designing bridges, soil structures, underground structures and buildings were proposed.

(2) Clarification of Design Procedure--Up to the present time earthquake resistant design has mainly been concentrated on the calculation methods. The fundamental design procedure has not satisfactorily been clarified. The Draft made clear the fundamental design procedure considering characteristics of earthquake ground motion, seismic performance and safety.

(3) Systematization of Design Procedure--Earthquake resistant design is required to have an adequate procedure in accordance with the characteristics of subgrounds and structures. The Draft shows fundamental conception for earthquake resistant design in order to indicate right procedure to follow.

Fundamental Conception of Aseismic Design

In earthquake resistant design following items should be considered.

(1) Characteristics of structures
(2) Uses of structures
(3) Types of structures
(4) Scales of structures
(5) Circumstances of structures
(6) Damages of structures and surrounding grounds caused by part earthquake

Fig.2 shows a fundamental procedure of aseismic design. In the procedure, the process to calculate seismic force or earthquake responce is classified into 4 methods as below.

(1) Seismic coefficient method
(2) Modified seismic coefficient method
(3) Seismic deformation method
(4) Dynamic analysis method

The method to be selected among these is determined by considering characteristics of structures.

Standard Seismic Loadings for Aseismic Design

Seismic zoning is presented herein in order to indicate characteristics of earthquake ground motion at each site. Frequency of past earthquakes, records of strong-motion earthquakes and seismic activity in a relatively wide area were taken into account in the zoning.

To determine the intensity of earthquake ground motion at certain site, seismic zoning as stated above and the characteristics of ground should be taken into consideration.

Seismic loadings for aseismic design is evaluated by the coefficients concerning the seismic zoning and the characteristics of ground according to the method of earthquake resistant design.

Soil Investigation and Seismic Behaviors of Subsoil Layers

Investigating the documents on the damages of soils caused by the past earthquakes is quite instructive in earthquake resistant design of structures. If it is necessary, the characteristics of soils at the construction site should be investigated. Seismic loadings are determined considering the results of investigations and examinations stated above.

The Draft also requires the examination and confirmation of soil stability.

Calculation Procedure for Building Structure

1. Design Procedures and Steps--This aseismic design method involves six design procedures and each procedure has two steps. One or more of the six procedures would be selected, according to the structural properties and the use of the building ; materials, seismic resisting system, height, etc.

The step 1 of these design procedures should satisfy the condition that the response of the structural members as well as the non-structural elements caused by the frequent earthquakes should not exceed the elastic range of the materials and the caused damages should be small enough that the repair would be scarcely required due to such earthquakes.

The step 2 following step 1 should satisfy the condition that the building structure would not collapse by the maximum possible earthquakes such as Kanto Great Earthquake in 1923.

The schematic chart of these design proceudres and steps is set forth in Table 1.

2. Seismic Force--The total lateral seismic force is basically determined by the following formula :

$$F = C \cdot W \tag{1}$$

Where C is the seismic design coefficient given by formalae (2a), (2b), (3a) or (3b). W is the total dead load of the building and applicable portions of other loads.

The seismic design coefficient for step 1 shall be determined in accordance with the following formula :

$$C_1 = Z \cdot Go(T) \cdot eCo \tag{2a}$$

Where Z is the seismic hazard zoning factor prescribed in section 3. Go(T) is the soil profile spectrum for approximation as given in section 4 and T is the fundamental natural period of the building in seconds. eCo = 0.2 is the standard base shear coefficient for step 1.

In procedure I-A or I-B, the height of the building concerned is low and the value of T is small. Accordingly, Go(T) is considered to be constant and 1.0. Therefore formula (2a) is reduced to (2b) :

$$C_1 = Z \cdot eCo \tag{2b}$$

The seismic design coefficient for step 2 shall be determined in accordance with either of the following formulae :

$$C_2 = Z \cdot Go(T) \cdot K_1 \cdot pCo \tag{3a}$$

$$C_2 = Z \cdot G\ (T) \cdot K_2 \cdot pCo \tag{3b}$$

3. Seismic Harard Zoning Factor--Seismic hazard zoning factor Z was determind by the seismicity, appling the theory of extremes, statistics and the engineering judgement. The values of Z are given in Fig. 3; seismic hazard zoning map.

4. Soil Profile Spectrum--The effects of the soil profile properties and the natural period of the building are expressed as the soil profile spectra as given in Fig. 4.

One of these spectra Go(T) is for the approximation and the rest four $G_1(T)$ correspond to four soil profile types.

The soil profile types are defined as follows :
Soil profile type 1 is a profile with rocks, hard and well consolidated sands, etc.
Soil profile type 2 is a profile with sands, stiff clays, hard loams, etc.
Soil profile type 3 is a profile which would not belong to other types.
Soil profile type 4 is a profile with soft clayes, loose sands, etc.

5. Structural Coefficient--It is a well known fact that the ductile structure could absorb more energy than the brittle one and it is not practical to design the building to be completely elastic when subjected to the maximum possible earthquake motion. Accordingly the structural coefficients, K_1 and K_2 are proposed and they would be denoted as the response modification factors dependent on the energy absorption capacity of the building to the earthquakes. The values of these factors are given in Table 2, according to the structural materials and system, and ductility level. K_1 would be used in procedure II-A and K_2 in procedure II-B, III-A and III-B, as shown in Table 1.

6. Seismic Hazard Exposure Factor--According to the use or the character of the occupancy of the building, the appropriate design procedure could be selected or the seismic hazard exposure factor could be adopted to be used in step 1 or 2. The value of the factor is not specified in this method, because the reliable theory to determine the value is not developed yet. But it can be suggested that the probability of failure, larger deviation of the seismic resistant capacity as well as far larger deviation of the seismic force, social utilities and the acceptable level of mortal risk should be studied to determine the value of the seismic hazard exposure factor.

Concluding Remarks

The proposed aseismic design method is based on the principle that make it possible to apply the new theory and research as well as to utilize the new materials and structural system. If one of the six design procedures would be selected properly, the building designed might be expected to stand against the strong motion earthquakes with economical designing effort and economical use of structural materials.

However, the details of the Draft might be subjected to minor modifications through the following extensive studies:

i) re-examination of various parameters
ii) case study on different types of buildings

iii) simplification of the Draft for practical use.

ACKNOWLEDGEMENT

The first half part of this report is taken from the original paper written by Dr. Toshihiko Hisada, Director of KAJIMA Institute of Construction Technology (Ref. 1), with some modifications to be appropriate for this workshop on Earthquake-Resistant Reinforced Concrete Building Construction

Here, the work by Dr. T. Hisada is deeply appreciated and acknowledged.

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Table 1 : SEISMIC DESIGN COEFFICIENT AND DESIGN METHOD ACCORDING TO DESIGN PROCEDURES AND STEPS

DESIGN PROCEDURE	SCOPE	STEP 1	STEP 2
I-A	CONVENTIONAL WOODEN BUILDINGS	$C_1 = Z \cdot eCo$ ELASTIC DESIGN WITH SEISMIC DESIGN COEFFICIENT AND ALLOWABLE STRESSES	STRUCTURAL DESIGN REQUIREMENTS
I-B	REINFORCED CONCRETE BUILDINGS WITH SHEAR WALL SYSTEM, etc.		
II-A	ORDINARY BUILDINGS	$C_1 = Z \cdot Go(T) \cdot eCo$ ELASTIC DESIGN WITH APPROXIMATE MODIFIED SEISMIC DESIGN COEFFICIENT AND ALLOWABLE STRESSES	$C_2 = Z \cdot Go(T) \cdot K_1 \cdot pCo$ PLASTIC DESIGN WITH APPROXIMATE MODIFIED SEISMIC COEFFICIENT AND APPROXIMATE ULTIMATE STORY SHEAR CAPACITY
II-B			$C_2 = Z \cdot G_1(T) \cdot K_2 \cdot pCo$ PLASTIC DESIGN WITH MODIFIED SEISMIC COEFFICIENT ULTIMATE STORY SHEAR CAPACITY
III-A	HIGH RISE BUILDINGS		$C_2 = Z \cdot G_1(T) \cdot K_2 \cdot pCo$ DYNAMIC DESIGN WITH MODAL ANALYSIS
III-B	SUPER HIGH RISE BUILDINGS		$C_2 = Z \cdot G_1(T) \cdot K_2 \cdot pCo$ DYNAMIC DESIGN WITH MODAL ANALYSIS AND TIME HISTORY ANALYSIS

C_1 : SEISMIC DESIGN COEFFICIENT FOR STEP 1
C_2 : SEISMIC DESIGN COEFFICIENT FOR STEP 2
Z : SEISMIC HAZARD ZONING FACTOR
Go(T) : SOIL PROFILE SPECTRUM FOR APPROXIMATION
$G_1(T)$: SOIL PROFILE SPECTRUM
T : FUNDAMENTAL NATURAL PERIOD OF BUILDING

eCo : STANDARD BASE SHEAR COEFFICIENT FOR STEP 1
pCo : STANDARD BASE SHEAR COEFFICIENT FOR STEP 2

K_1 : STRUCTURAL COEFFICIENT FOR APPROXIMATION
K_2 : STRUCTURAL COEFFICIENT

Table 2 : STANDARD STRUCTURAL COEFFICIENT

	STRUCTURAL COEFFICIENT	DUCTILITY LEVEL		
		A	B	C
REINFORCED CONCRETE BUILDINGS PRESTRESSED CONCRETE BUILDINGS	K_1	0.4	0.5	0.6
	K_2	0.3	0.4	0.5
STEEL REINFORCED CONCRETE BUILDINGS	K_1	0.3	0.4	0.5
	K_2	0.3	0.3	0.4
STEEL BUILDINGS	K_1	0.3	0.5	0.6
	K_2	0.3	0.4	0.5
WOODEN BUILDINGS	K_1		0.4	
	K_2		0.3	

DUCTILITY LEVEL A : DUCTILE BUILDINGS
DUCTILITY LEVEL B : INTERMEDIATE BUILDINGS
DUCTILITY LEVEL C : BRITTLE BUILDINGS

1) Steel Structure
2) Modified on the Basis of Japanese
   Allowable Stress Yield Point

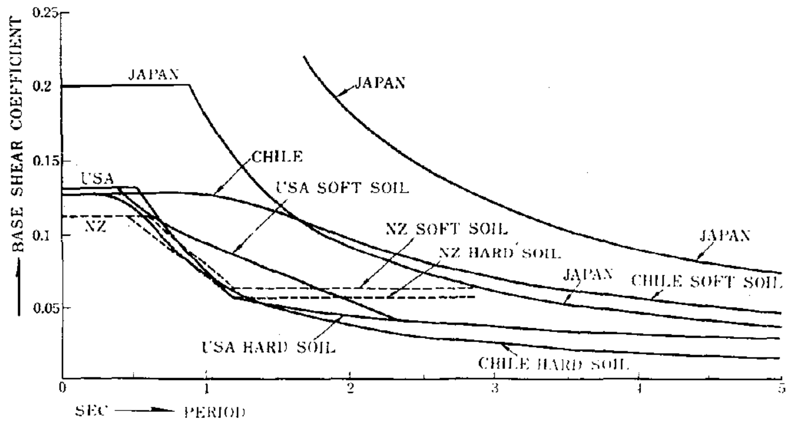

Fig. 1 Comparison of Base Shear Coefficients

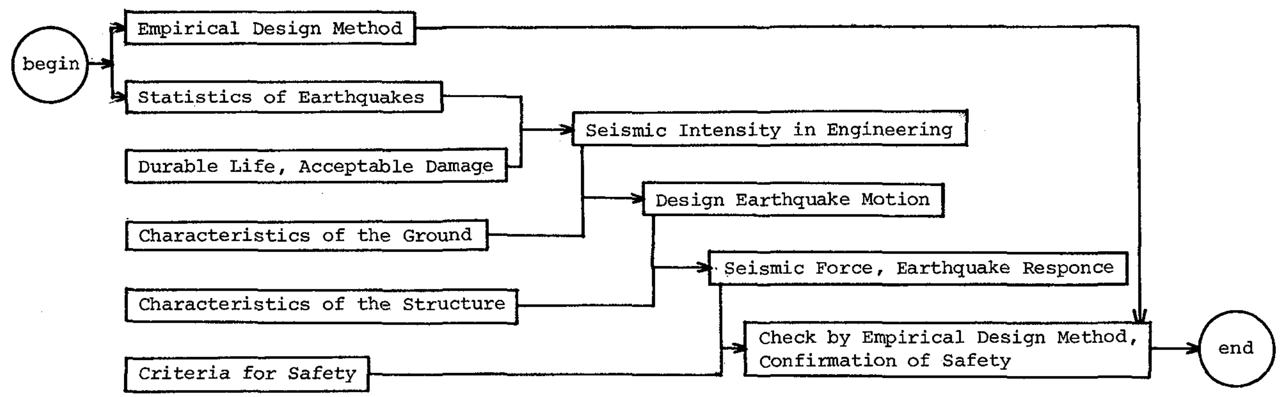

Fig. 2 Basic Procedure for Earthquake Resistant Design

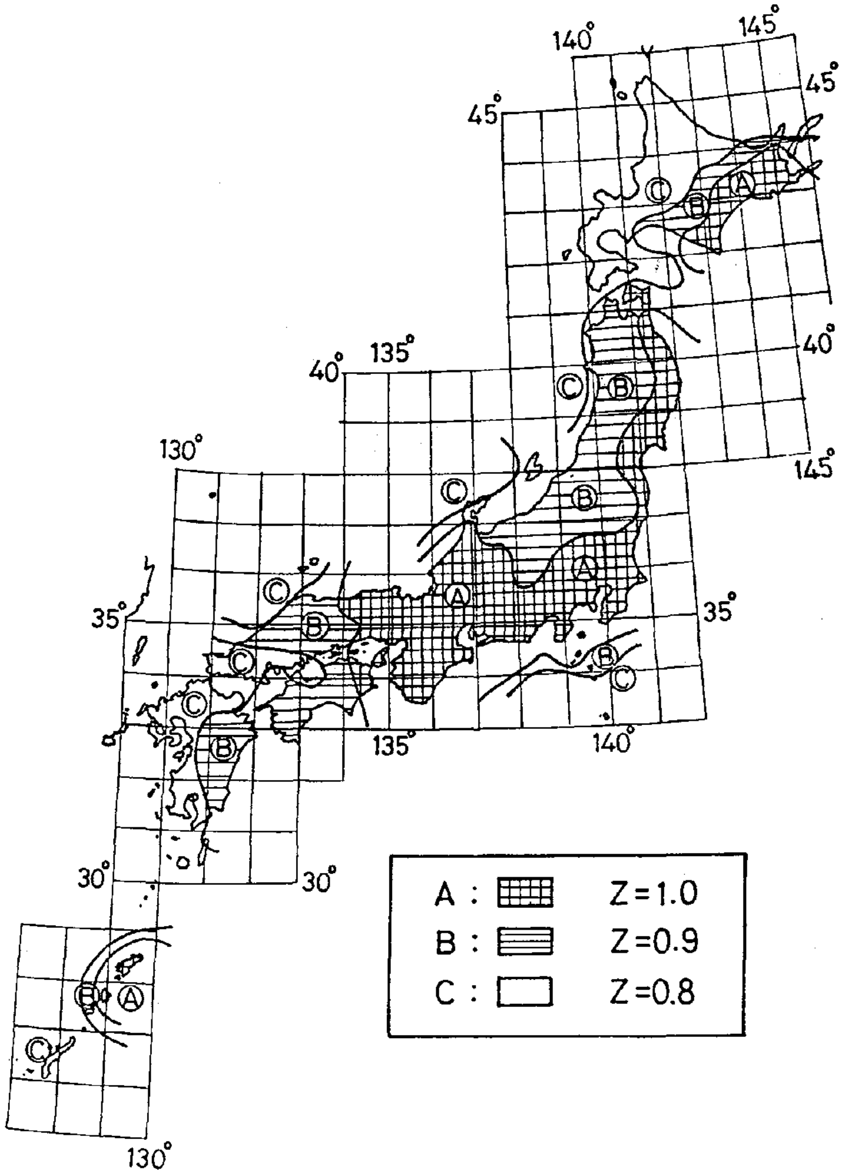

Fig. 3 Seismic Hazard Zoning Map of Japan

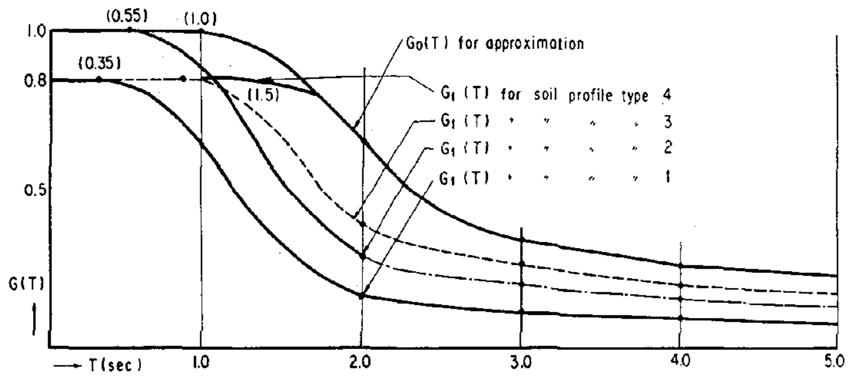

Fig. 4 Soil Profile Spectra

SEISMIC CODE BASED ON SEMI-PROBABILISTIC APPROACH

by

Jack R. Benjamin
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ABSTRACT

The paper presents a rapid overview of the techniques currently employed in the probabilistic forecasting of earthquake ground motions and a brief introduction to reliability principles as they could be applied to reinforced concrete construction. It is shown that while the basic forecasting methodology is reasonable, the techniques leave much to be desired particularly with respect to the assessment of criteria.

Several new developments are presented which show promise of increasing the state of knowledge about ground motion from an improved treatment of existing data. The basic error in response spectrum code provisions is discussed. A limited statistical analysis of the response characteristics of oscillators to a variety of earthquakes is presented as well as the results of decomposition of records into collections of transients.

INTRODUCTION

The purpose of this discussion is to present a rapid overview of a very large subject. The results of probabilistic analyses that appear as functions or numbers in seismic codes are few and it is unlikely that this will change greatly in the next few years. There is a natural reluctance on the part of professional engineers to change a subjective design practice on the basis of the mathematical rationality of a poorly understood procedure.

A code design criteria must be deterministic and each such item is the result of a decision process. In contrast, the occurrence of earthquakes and associated response of structures to earthquake ground motion is uncertain prior to the occurrence of the event. Such scientific forecasts about the uncertain future, however, are one of the key ingredients in the decision process which results in the code. This discussion is primarily concerned with the making of the rational probabilistic scientific forecasts that can be reflected in criteria.

The material that follows focuses on loadings since the associated methodologies have become relatively stabilized. In contrast, although a great deal is known about the real performance of reinforced concrete members and assemblies, this information has not been properly translated into codes. The interaction between the variabilities of strengths in assemblies has not been considered by code-making bodies. The theory of reliability has not been considered in codes dealing with reinforced concrete. Reliability theory differentiates between precast and continuous cast-in-place construction, with the latter being given a position of advantage that it does not have in the current codes.

The techniques that follow have evolved over a considerable span of time and are continuing to slowly evolve. Standardization of procedures has had the usual result of inhibiting research. The developments have occurred in diverse fields under a multitude of differing circumstances ranging from the need for an academic paper to the acceptable risk associated with a nuclear power plant. In addition, scientists have often considered themselves to be engineering-decision makers, thereby confusing their recommendations as scientists.

Forecasting of Earthquake Loads

The probabilistic forecasting of future earthquakes and associated consequences follows a simple and direct logic. The basic assumption is that recorded history provides an adequate basis for forecasting the future. Thus, if the ground at the site of interest has shaken with an intensity of MMI VIII on an average of once every 50 years for a sufficiently long period of time, the assumption is made that future MMI VIII events will occur at average 50-year intervals. Aside from uncertainty as to the design requirements for a MMI VIII intensity, such as forecast is both useful and reasonable.

Poisson Occurrence Forecasting

Before examining the more complex cases of seismic source areas, attenuation, levels of events, ground shaking measures, and decision making, it is useful to continue the previous example. It was assumed that only MMI VIII events are of interest and one such event occurs at the site each 50 years on a long-run average. For example, assume that the times in years between the last five events are:

	35 years	
	50	
	45	
	70	
Sum	200	Mean = 50 years

The quantity of data is small and the scatter is large so that it is difficult to justify a very complex model of occurrence based on strain accumulation, for example, even though the tectonic evidence indicates this could be the case.

The simple Poisson process [1],[2] is selected to model the occurrence of MMI VIII earthquakes at this site. The assumption is that an MMI VIII earthquake is equally likely to occur in any year (despite the fact that very short time intervals are not found in the data). If the probability of an event in any year is one in fifty or 0.02, the occurrences are simple Poisson. This is the basic model employed in all practical forecasting although much more sophisticated models could be used.

The probability of r events in t years if the annual mean rate of occurrence is u is defined by:

$$p(r) = \frac{e^{-ut}(ut)^{r}}{r!}$$

The mean of r is: $m(r) = ut$

If $u = 0.02$ and $t = 50$, $m(r) = 1$. The return period or recurrence interval of MMI VIII events is 50 years. The probability of occurrence of an event with a 50-year return period in the period 0 to 50 years is 0.63 or almost two-thirds.

The usefulness of this model is readily shown. If the probability of occurrence in any year is u, the probability of nonoccurrence in any year is 1-u, and the probability of n consecutive years of nonoccurrence is $(1-u)^n$.

Thus the probability of at least one occurrence in n years is,

$$1 - (1-u)^n$$

so that if $u = 0.02$, $n = 50$. (Forecast time is the same as return period.)

$$P(\text{at least one occurrence}) = 1 - 1(-0.02)^{50} = 0.63$$

From the Poisson model,

$$\begin{aligned} P(\text{at least one occurrence}) &= 1 - P(\text{no occurrences, } r = 0) \\ &= 1 - e^{-ut} \end{aligned}$$

This expression is very useful in criteria studies. If the useful life of the facility is 50 years and we are willing to accept a probability of 10 percent that at least one criteria or larger event will occur in that 50 years, the design event is that having a 475 year return period since

$$0.10 = 1 - \left(1 - \frac{1}{475}\right)^{50} = 1 - e^{-\frac{50}{475}}$$

This is the basis of the recently published seismic hazard maps by Algermissen and Perkins of the USGS [3], as well as the ATC-3 maps of effective peak acceleration and velocity.

Note the value of the concept in comparing different geographical areas since a 500-year event can involve different levels of events. That is, if the 500-year events at differing localities are used to design two structures (different designs) and losses are identical with exceedance of this load level, the mean annual losses are identical at the two sites.

Levels of Occurrence

As a first approximation, many natural phenomena show an exponential relationship between size of event and event frequency. This type of multiplicative relationship betweeen size and frequency appears to fit earthquakes small to large in size, excluding both very small events and great events.

A plot of intensity or magnitude against the log of frequency of exceedance gives a reasonable fit to a straight line. It has been common to neglect uncertainty in magnitude and thus to fit a straight line to plotted points assuming that the variability is entirely confined to frequency of exceedance. Such a plot defines the mean annual rate of occurrence of events of sizes within the data and thus fits into the discussions of the previous section.

As long as such a plot was only an issue in academic discussions, there were few problems. However, the extension to criteria and codes has produced serious unresolved problems of two types. First, what is the maximum credible event and, second, how does the code handle the more frequent lower level event? The Nuclear Regulatory Commission's (NRC) Safe Shutdown Earthquake (SSE) and Operating Base Earthquake (OBE) are examples of these problems. The NRC regulations define the OBE as one-half the SSE, thereby implying a linear relationship to a highly nonlinear problem.

It is thus worthwhile to examine the level of occurrence problem in some detail. Assume that the data are in the form of Modified Mercalli Intensity (MMI) for 135 years of historical record. The year of occurrence is unimportant since these are Poisson events. In 135 years, the record is:

Intensity	Number
VII to VIII	1
VI	4
V	7
IV to V	1
IV	30

These data are plotted in Figure 1 as Source 1 according to the assumption that the events are Poisson and intensity is exponentially distributed. The data are plotted by beginning with the largest event, VII.5 (average of VII and VIII), and plotting this value at a 135-year return period or 0.0074 annual exceedance probability. The next data point is the first VI event plotted at a return period of 135/2 = 67.5. The next VI event is plotted at 135/3 = 45 years, the fourth at 135/4 = 33.7 years, etc. The plot on semi-

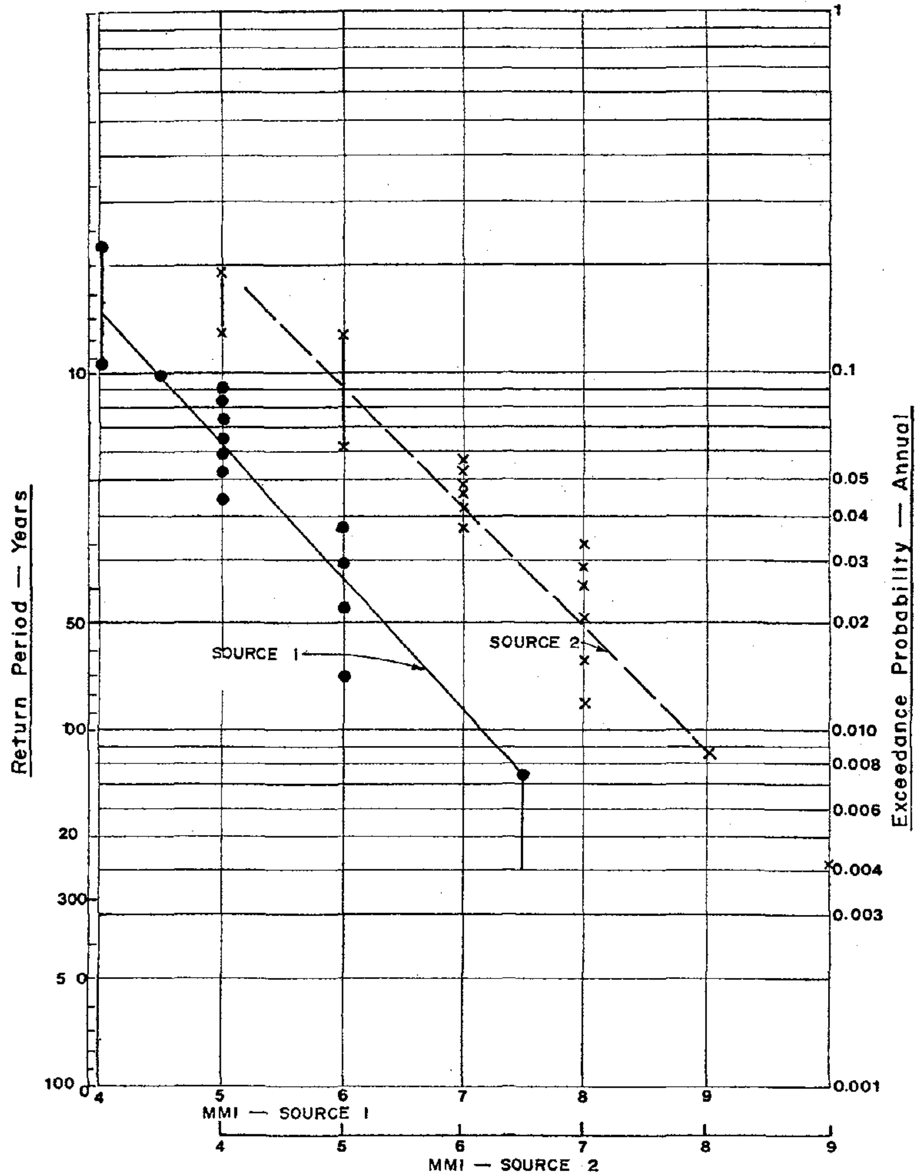


FIGURE 1 HISTORICAL DATA - SOURCE 1 AND 2

log paper shows a reasonable fit to a straight line. A second set of data is shown for Source 2 for the purposes of comparison. Source 2 differs tectonically from Source 1 but is in the same general geographic region.

The assumption is that the probability density function of intensity is as shown in Figure 2a. Unfortunately, the assumed model is unlimited with respect to large values of intensity. Following conventional procedures, the assumption is then made that the maximum credible event is VII.5 or a larger value. If the largest possible event is VII.5, this effectively says that the model is bilinear on Figure 1 as shown and the model is that of Figure 2b. The model of Figure 2b is not compatible with our knowledge of physical phenomena in that the probability density function cannot contain discontinuities. For this reason, two more assumptions were investigated, limits at VIII and IX with models shown in Figures 2c and 2d.

Is the assumption of large limits more or less conservative? Table 1 shows that the rough model is more conservative as long as the theory of probability is applied properly. Difficulties have arisen, however, as a consequence of selective application of probability concepts such as a neglect of normalization. If the tail of a probability distribution is truncated or cutoff, it is necessary to normalize the model to make the area unity again.

Thus it is possible to examine various assumptions rationally with respect to largest values. It is also possible to examine the lower level event defined on the basis of allowable damage or the limit of elastic response, for example. Although such concepts have only been explored, it is possible to construct a loss function for a set of graded designs against earthquake intensity. Each of these loss functions can be combined with the occurrence-level relationship to obtain an expected annual loss from earthquakes for each design. A cost-effective analysis follows directly with the result being identification of the optimum design considering all possible future earthquakes. One obvious difficulty with such analyses is the interaction between the structural systems and other systems in terms of function of the facility as well as hazards. That is, all systems must be considered, not just those associated with losses. Losses by themselves do not constitute a basis for decision making. The motivation for construction of a building is not the avoidance of earthquake losses.

Sources and Attenuation

Earthquakes are rare occurrences so that the possibility of simultaneous great events on the San Andreas and the Hayward faults can be neglected. Thus it is possible to consider several sources by simple addition of separate influences. In general, near field earthquake effects are not properly considered in existing seismic risk analysis procedures. Thus the site of interest should be at some distance from the energy source in order for the methodology to be valid. Obviously, this is not a problem if the data consist of intensity or ground shaking characteristics at the site of interest from diverse earthquakes. In the absence of such data, the technique is effectively to simulate such a set of data by attenuating the ground motion from earthquake sources to the site in question with each such source associated with an appropriate annual frequency-size relationship.

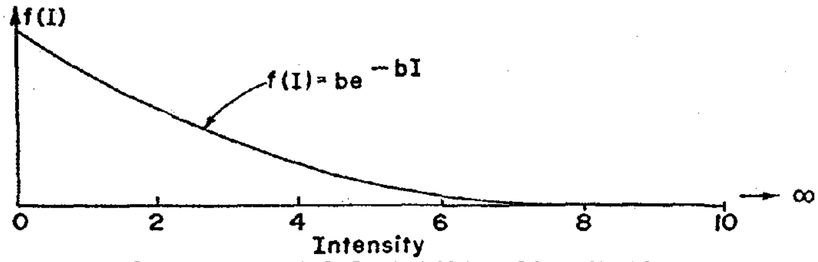


Figure 2a Exponential Probability Distribution

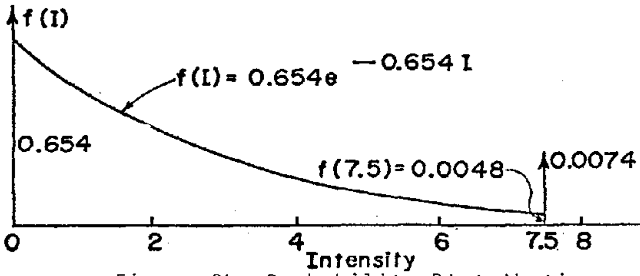


Figure 2b Probability Distribution

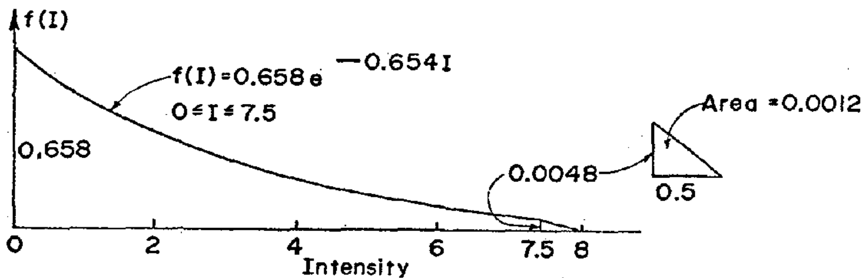


Figure 2c Probability Distribution

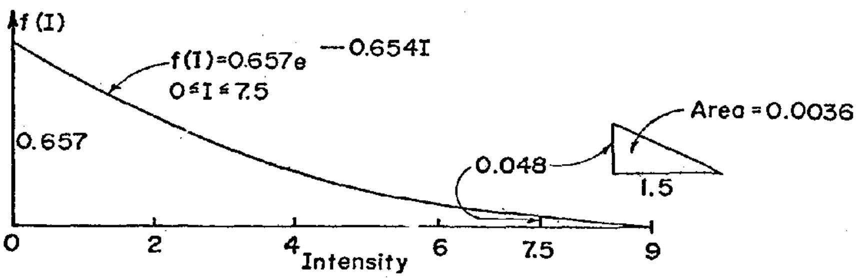


Figure 2d Probability Distribution

FIGURE 2 PROBABILITY DISTRIBUTIONS

TABLE 1

ANNUAL EXCEEDANCE PROBABILITIES AND RETURN PERIOD

	LINE 1 (Bilinear)		LINE 2 (Smoothed to MM VIII)		LINE 3 (Smoothed to MM IX)	
MM	Probability	Return Period In Years	Probability	Return Period In Years	Probability	Return Period In Years
V	0.0380	26	0.0320	31	0.00332	30
VI	0.0198	51	0.0137	73	0.01499	67
VI.5	0.0142	70	0.00809	124	0.00927	108
VII	0.01027	97	0.00413	242	0.00532	188
VII.5	0.00740*	135*	0.00126	796	0.00245	409
VIII			0	∞	0.00112	896
VIII.5					0.00027	3750
IX					0	∞

*Probability and Return Period for MM VII.5 event.

Errors unavoidably exist for sites located inside major seismic energy sources that can only be resolved by employing a finer net of source areas. These errors are associated with the assumption that every point along a selected fault length has equal likelihood of being an epicenter or every point in a source area can be an epicenter location. With large source areas or long faults, this can be a gross approximation for sites near to the energy source since it amounts to the shifting of energy from areas of high-energy-release rates to areas actually having lower levels of activity. Yet there is not much choice in modeling owing to the extreme unlikelihood of an exact repetition of history plus strain-build-up considerations that make areas with little historic record of activity likely sources of future activity.

Attenuation can be in terms of Modified Mercalli Intensity or magnitude. It is indicative of the state of knowledge that each of several forecasting techniques can produce up to an order of magnitude difference in site forecasts. For example, if an M 8.25 earthquake is assumed on the San Andreas fault and the site of interest is the Berkeley campus of the University of California, it is possible to attenuate M 8.25 directly to the site in terms of peak ground acceleration. Alternatively, the M 8.25 event can be related to acceleration at the fault and then this acceleration attenuated to the site. The M 8.25 can also be translated to MMI at the fault, MMI attenuated to the site and then the site MMI translated into acceleration. The range of forecasts by various techniques can involve as much as a factor of 10 in the final results and yet each of the relationships used makes best use of the available data.

The reason for extreme variability or uncertainty is usually the result of the combination of systematic variabilities with random effects. This is the case here. In the forecasting problem, the models used are quite obviously gross approximations of very complex phenomena. It is possible to make more refined studies but at great increase in cost and effort. Very sophisticated methodologies and techniques exist that can resolve many of these issues but thus far there has been no government or industry support for such investigations. The support issue is complicated by uncertainty about cost effectiveness of such work.

Ground Motion at the Site

From the standpoint of structural analysis and design, the earthquake requirements can be expressed in three different forms, each with many variations. A simple static coefficient may be employed which has no direct relationship to probabilistic methodology other than it being the result of a committee vote made under uncertainty. Unfortunately, geologists and seismologists have interpreted such a judgmental coefficient in terms of peak instrumental ground acceleration. The apparent direct relationship between the ATC-3 effective peak acceleration map and the Algermissen-Perkins USGS map [3] is the result of a desire to avoid such criticism and misunderstanding while retaining more or less the same basic design levels by adding sets of balancing coefficients in the engineering design recommendations.

Response spectra have received a somewhat different treatment again with the discontinuity between science and criteria in keeping with their differing functions. The response spectrum for one particular record is shown in Figure 3. If now attention is focused on structures with a period of less than 1 second more or less, all response spectra of interest are similar in shape, concave downward, and all show such extreme variability that it is difficult to select either a mean value function or reasonable engineering envelope of maximum values. Furthermore, it appears that response spectrum ordinates associated with large peak instrumented ground accelerations are larger than those with small peak instrumental ground accelerations. In the absence of a better normalizing constant, it has been assumed that instrumental records and response spectra can be scaled on the basis of peak instrumental ground acceleration. Records have also been scaled similarly despite obvious conflicts with the physics of the phenomena such as size of the energy source. That is, each earthquake is a separate individual event rather than a scaled sample of a single master event.

If we examine a set of peak ground acceleration (PGA) normalized response spectra for a variety of earthquakes at similar instrument sites, a figure such as that shown in Figure 4 is obtained. Now if the individual normalized spectral ordinates at each frequency are assumed to be independent random variables, it is possible to obtain the mean and standard deviation of the data set at each period, and even fit a probability distribution to the data. The criteria shape is then related to say the mean value plus one standard deviation after smoothing the results and simplifying the form for purposes of criteria establishment. This is the essential basis of the standard criteria response spectra.

Unfortunately, a fundamental error exists in the analysis leading to the criteria. The assumption is made that normalized spectral ordinate values at every frequency are independent random variables when, in fact, all spectral ordinates for a given earthquake are not independent but dependent. The normalizing constant itself, PGA, is the realization of one particular random variable; thus it is no more a valid normalizing constant than any other realization. If there is any independence, it exists between earthquakes and thus can be questioned too. A completely different method of statistical analysis should have been used. The proper technique is to systematically model each response spectrum for each earthquake and then compare the values of model parameters (Fig. 5). There exists no apparent interest in such studies since the results may well conflict with established criteria.

The most valid design criteria is the recorded ground motion from a perfect instrument at the site for the design earthquake event. Since this ideal cannot be attained, the alternative is to select a set of ground motions from nearby instrumental records for a representative group of earthquake events. A level of uncertainty is introduced since the future may differ from the past and some weighting is needed in interpreting the results. The weighting is actually an expression of Bayesian probabilistic concepts.

Two contrasting techniques exist in arriving at code-related loading time histories, the actuarial approach and the artificial time history approach. If a number of instrumental records are compared in a gross sense, the overall impression is that of a lack of order in appearance. This appar-

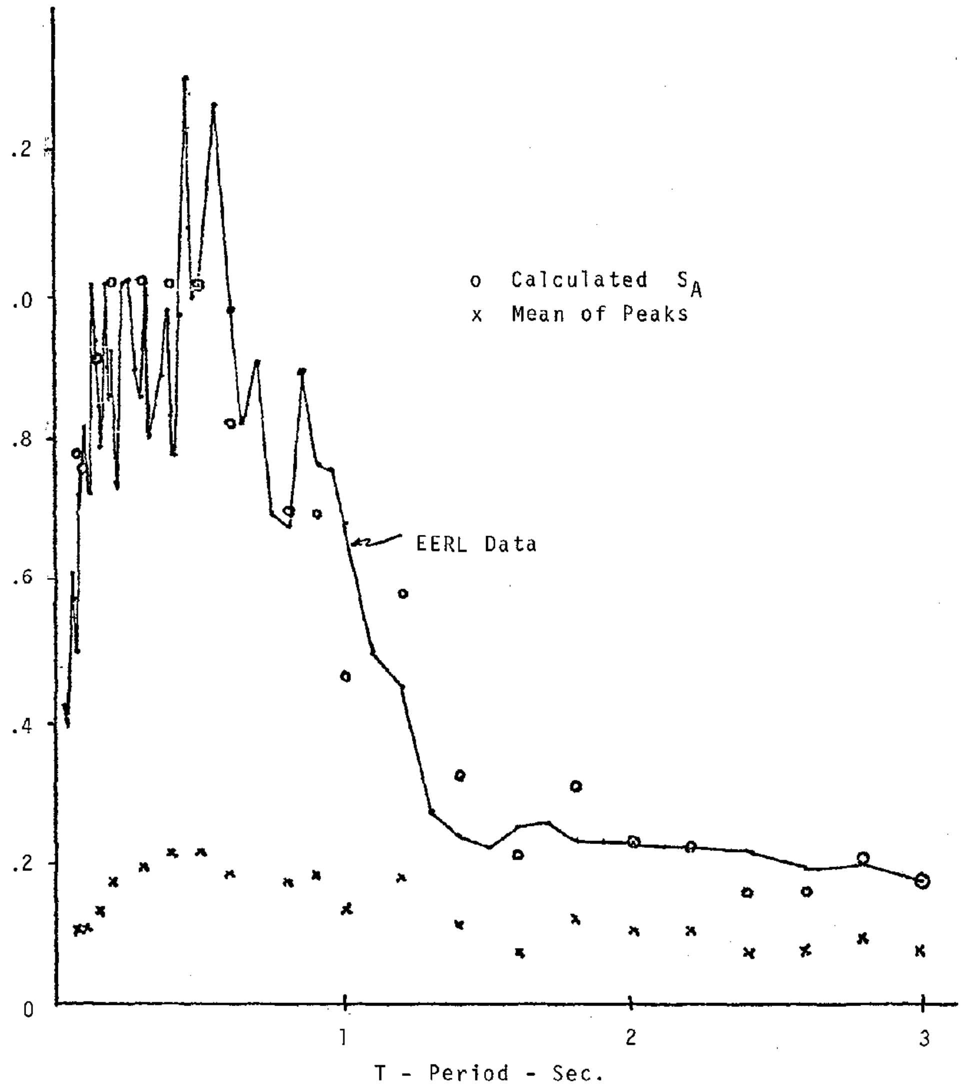


FIGURE 3 RESPONSE SPECTRUM

1940 EL CENTRO SOOE, 2 PERCENT

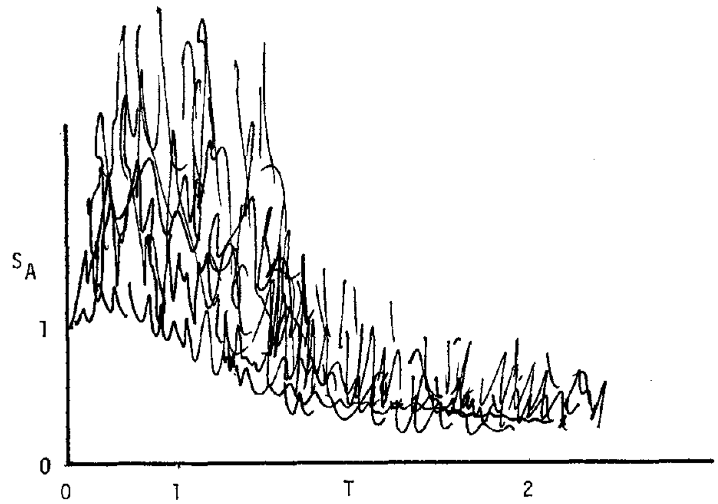


Figure 4a Data Plot

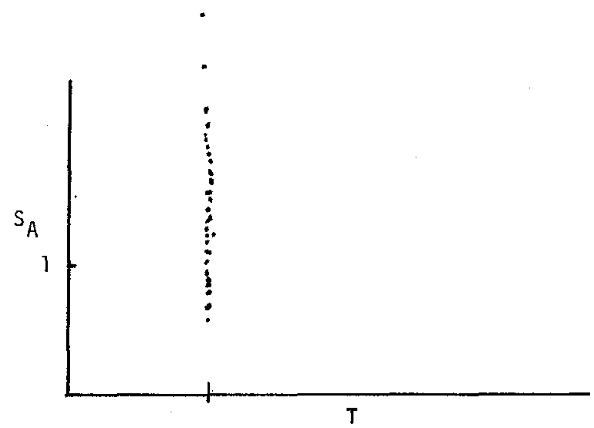


Figure 4a Data at One Period

FIGURE 4 RESPONSE SPECTRA ANALYSIS

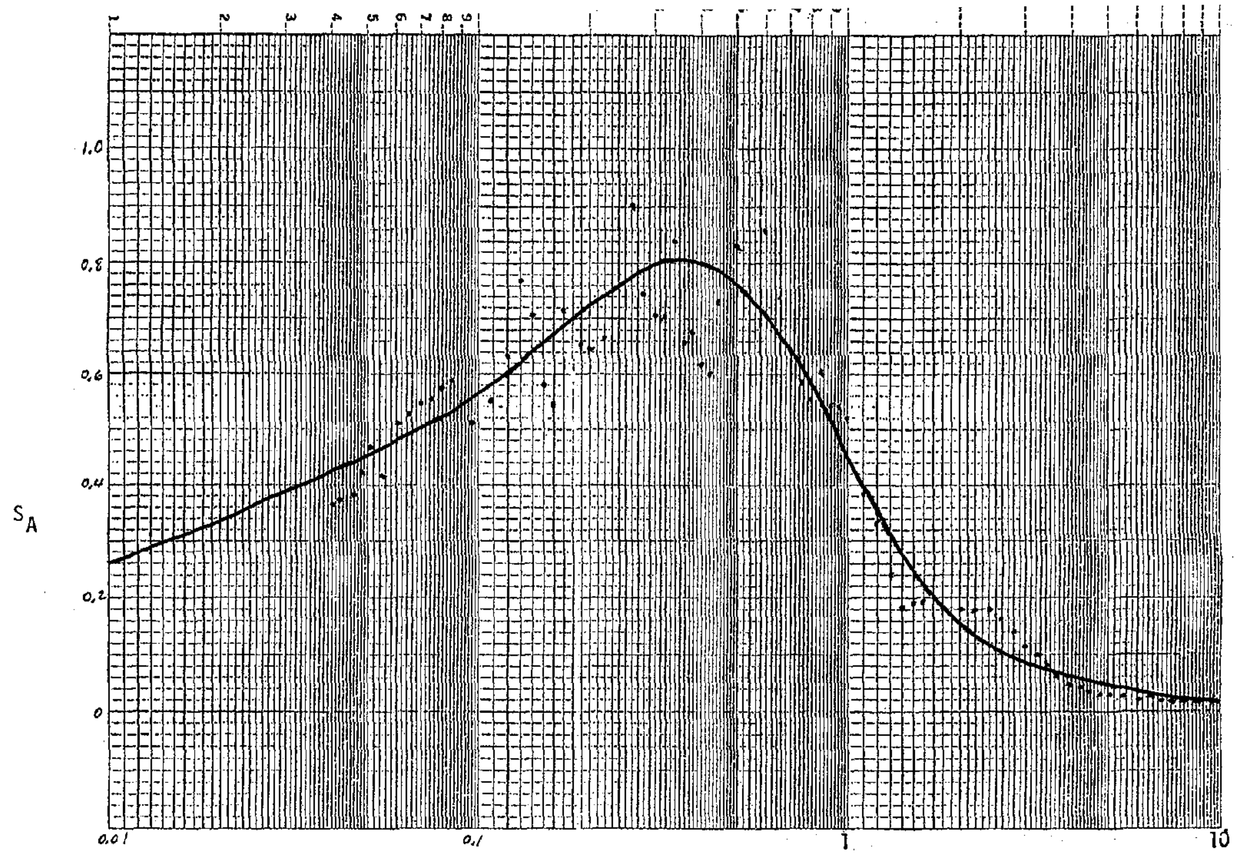


FIGURE 5 STATISTICALLY FITTED RESPONSE SPECTRUM,
EL CENTRO 1940, SOOE, 5 PERCENT DAMPING

ent lack of order has been considered to be the result of randomness, and visual comparison with random vibrations leads to the conclusion that random vibration theory affords a useful model for earthquake ground motion. Although the random vibration model is probabilistic, the fact that the probability model is deterministically prescribed changes it from the more general probabilistic approach in which there is uncertainty about model and parameters to a pragmatic model with engineering utility. Time histories artificially generated to fit a prescribed response spectrum fall in the same classification.

In sharp contrast, the actuarial approach to probabilistically prescribing time histories essentially focuses on response, which in turn leads to modeling of the aspects of earthquake ground motion that directly bear on response. The simplest way to understand this methodology is to examine the response time histories of linear mass-spring-dashpot systems to particular earthquakes. Figures 6 through 11 are samples of these time histories. At first glance, it appears that the models respond at their theoretical periods of vibration, although the input motion contains energy at all periods. Upon closer inspection, numerous phase shifts can be recognized. The Pacoima Dam response illustrates the great shift in response pattern in time with changes in periods of the responding model. In fact, for periods of one and two seconds, the response is almost as though the earthquake were a single pulse of energy after which the model response is very similar to damped free vibration. The response pattern with the El Centro records differs only in detail.

Now if statistical studies of response time histories are made for a wide variety of periods and 12 records from several earthquakes (Fig. 12 and 13), it is found that every response time history appears to have almost the same statistical properties independent of earthquake and record.

Finally, a technique has been developed for exact decomposition of digitized records into a set of wave trains. Each wave train has a prescribed period, but amplitude and phase are functions of time. The partial result of such a decomposition is shown in Figure 14. It is now possible to identify the motions that produce maximum response for any vibrating system, linear or nonlinear. The statistical analysis of the separate wave trains for differing earthquakes and records shows great uniformity in properties. It appears to be in the realm of possibility to obtain a rather simple probabilistic model of the entire ground motion that will fit all ground motion records satisfactorily and contain only a few parameters.

RELIABILITY

During the last 15 years a great deal of effort has gone into research leading to probabilistic models of strength and rigidity of reinforced concrete members and assemblies. The individual member problem is virtually solved from a theoretical viewpoint. Vast literature exists on this subject. Outside of areas influenced by ACI 318, a first start has been made by using characteristic values coupled with sets of factors of various types, many of which are based on the statistical properties of members.

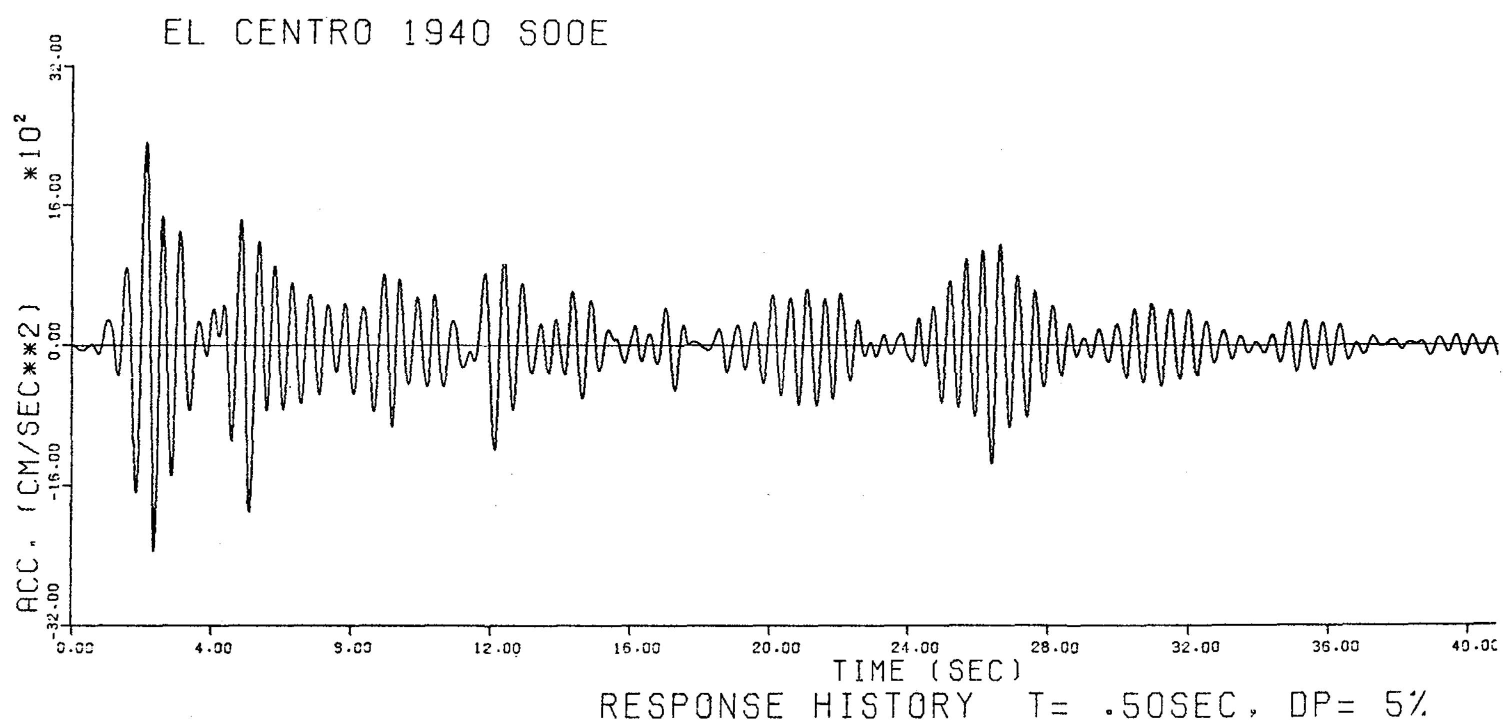


FIGURE 6 RESPONSE HISTORY EL CENTRO 1940 SOOE, T = 0.5 SECONDS

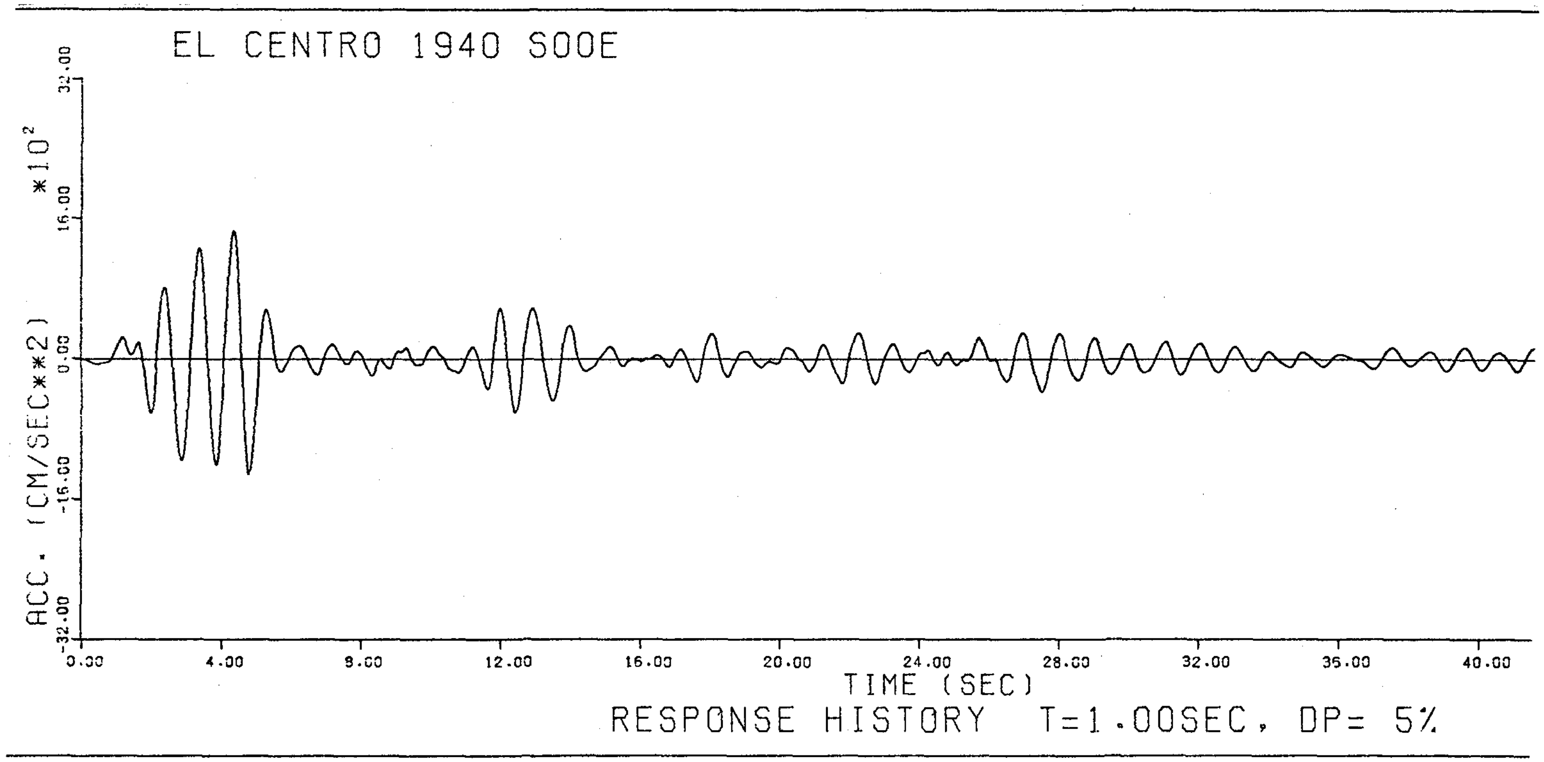


FIGURE 7 RESPONSE HISTORY EL CENTRO 1940 SOOE, T = 1.0 SECONDS

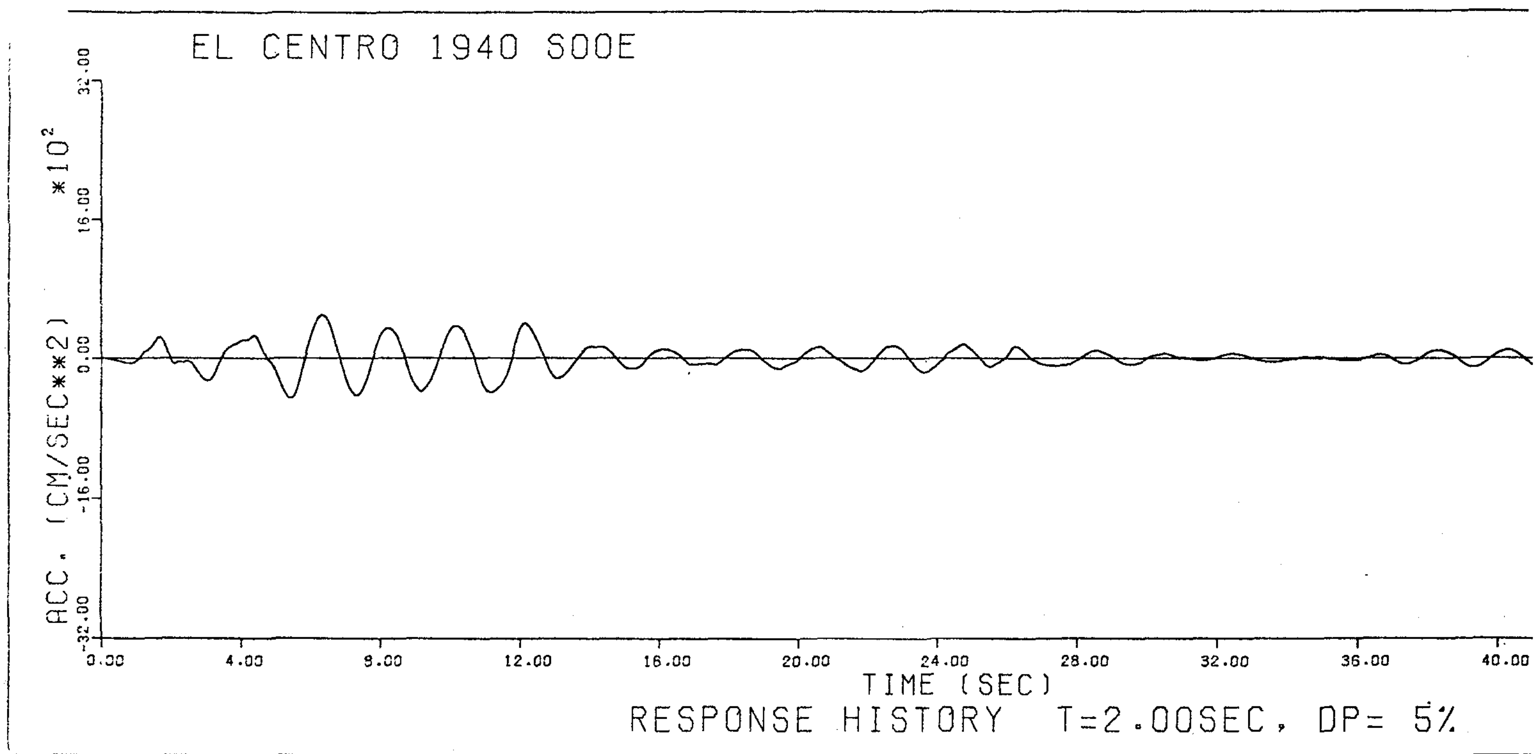


FIGURE 8 RESPONSE HISTORY EL CENTRO 1940 SOOE, T = 2.0 SECONDS

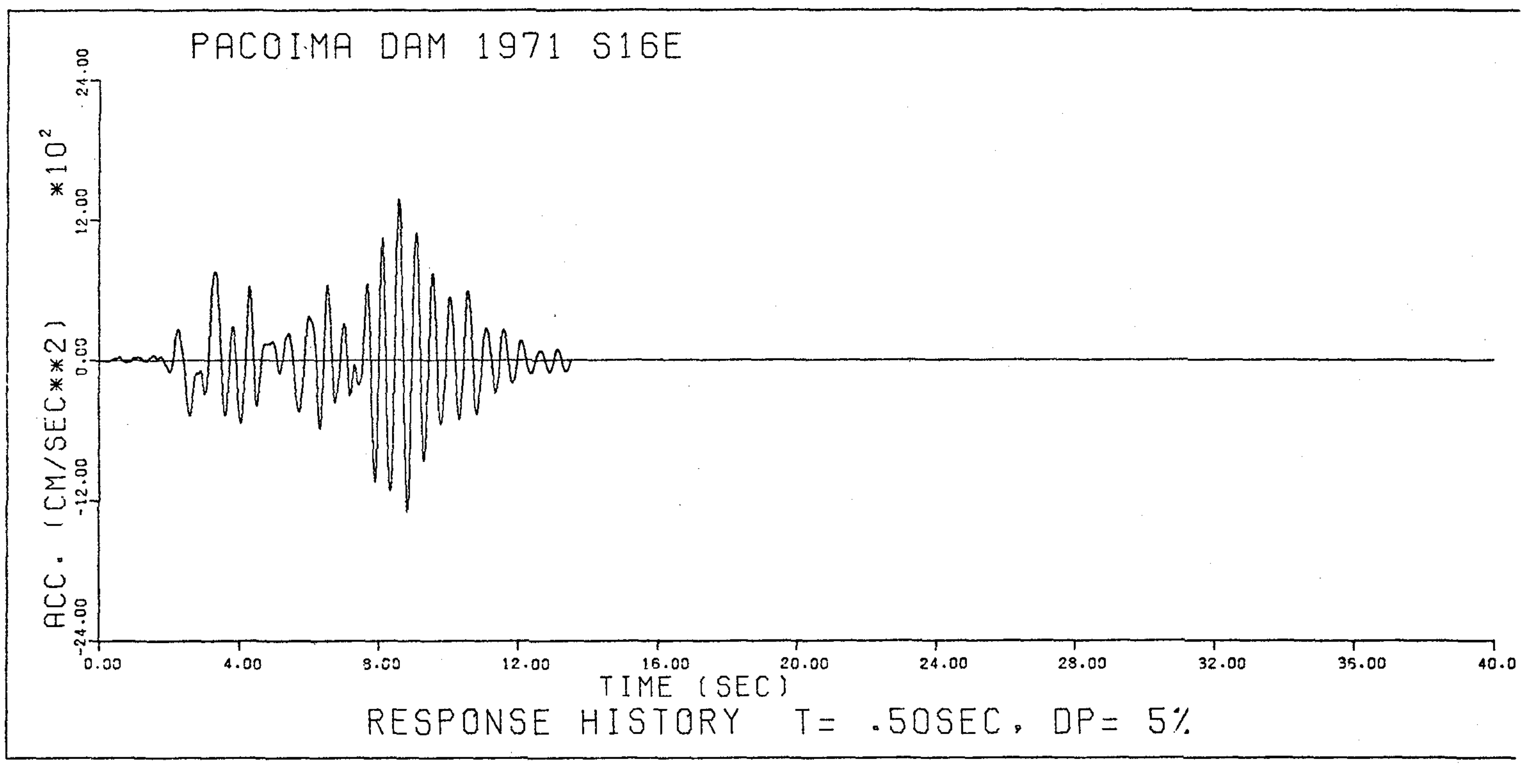


FIGURE 9 RESPONSE HISTORY PACOIMA DAM 1971 S16E, T = 0.5 SECONDS

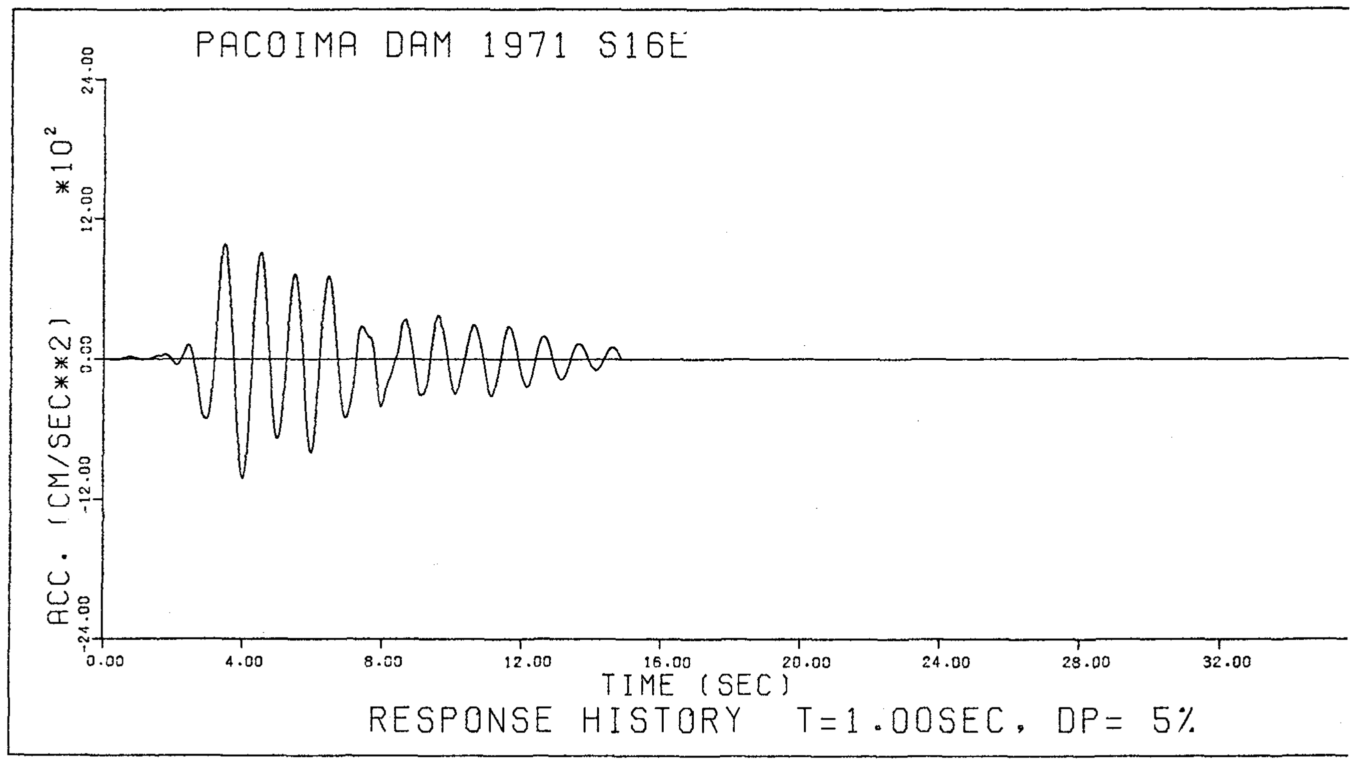


FIGURE 10 RESPONSE HISTORY PACOIMA DAM 1971 S16E, T = 1.0 SECONDS

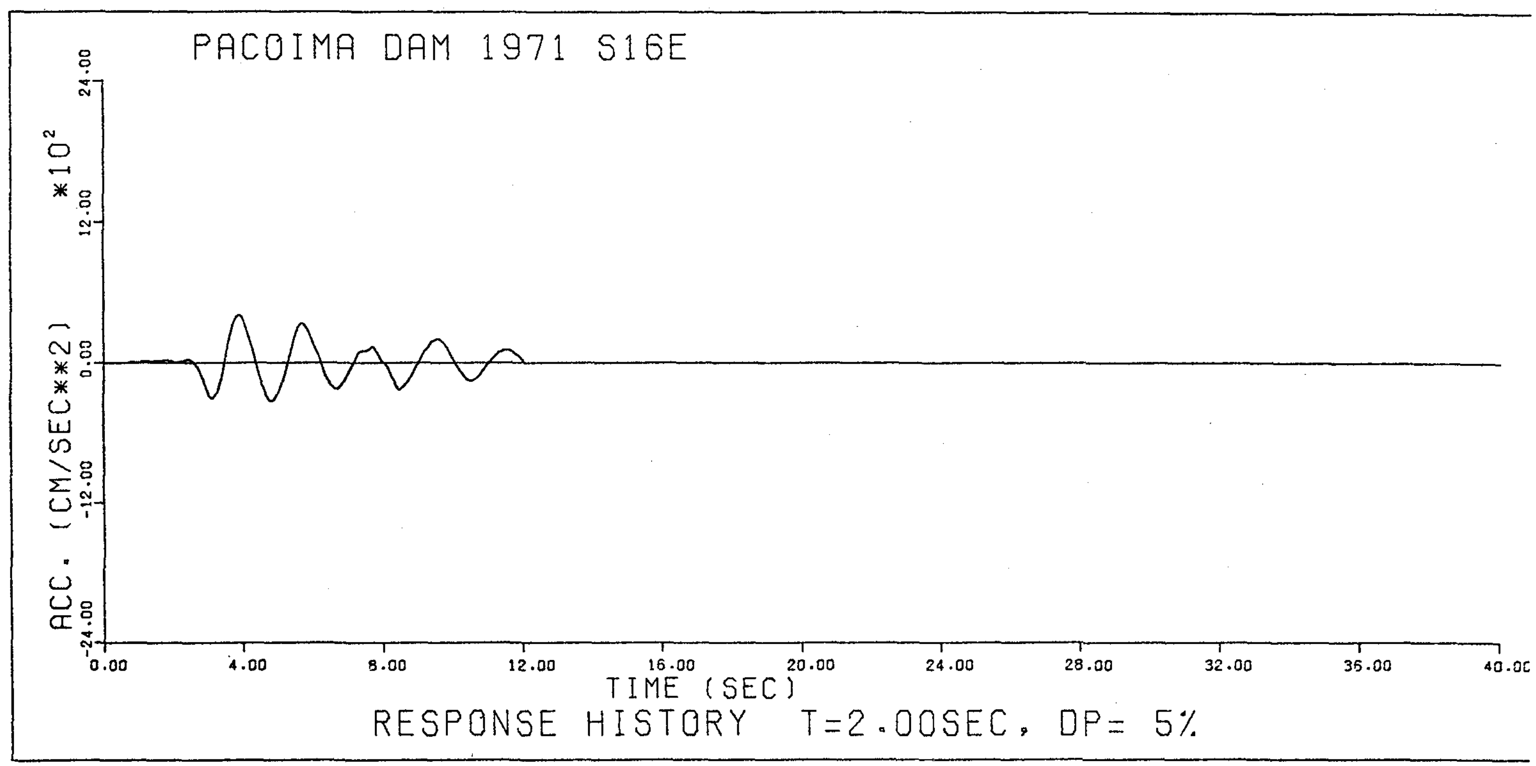


FIGURE 11 RESPONSE HISTORY PACOIMA DAM 1971 S16E, T = 2.0 SECONDS

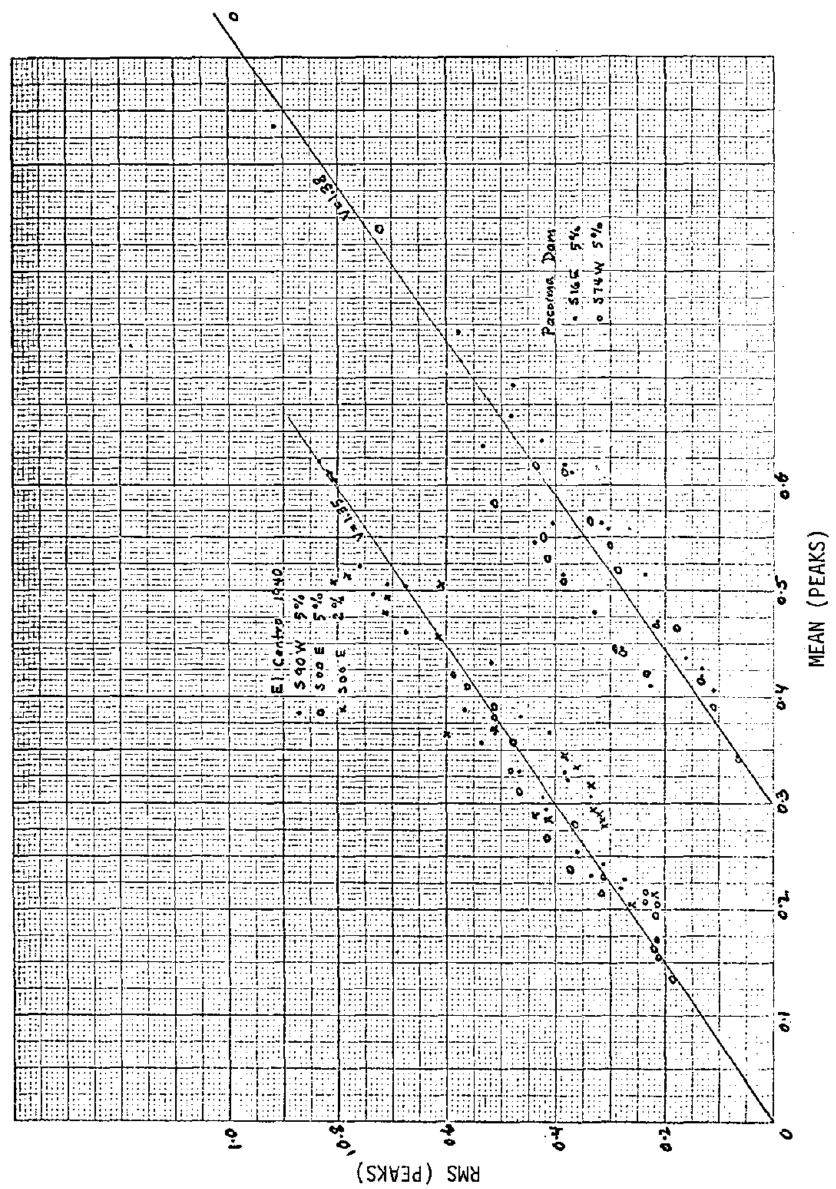


FIGURE 12 RMS OF PEAKS VERSUS MEAN OF PEAKS

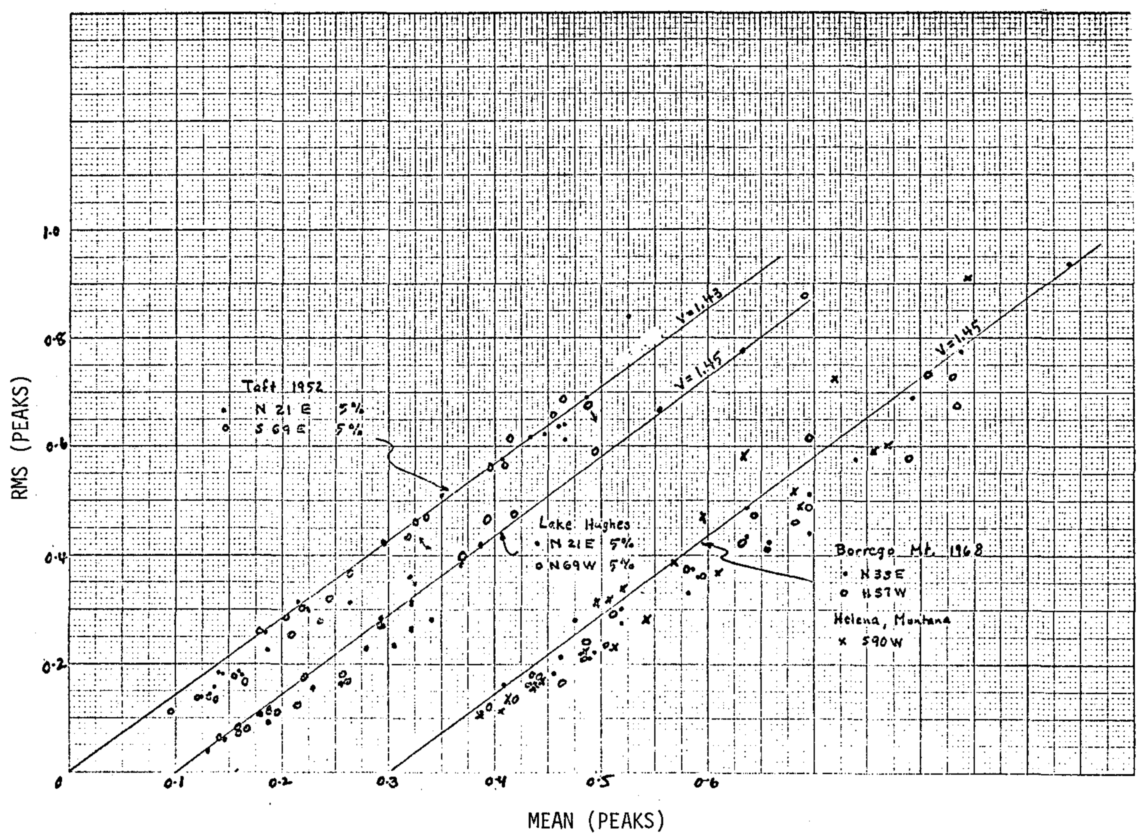
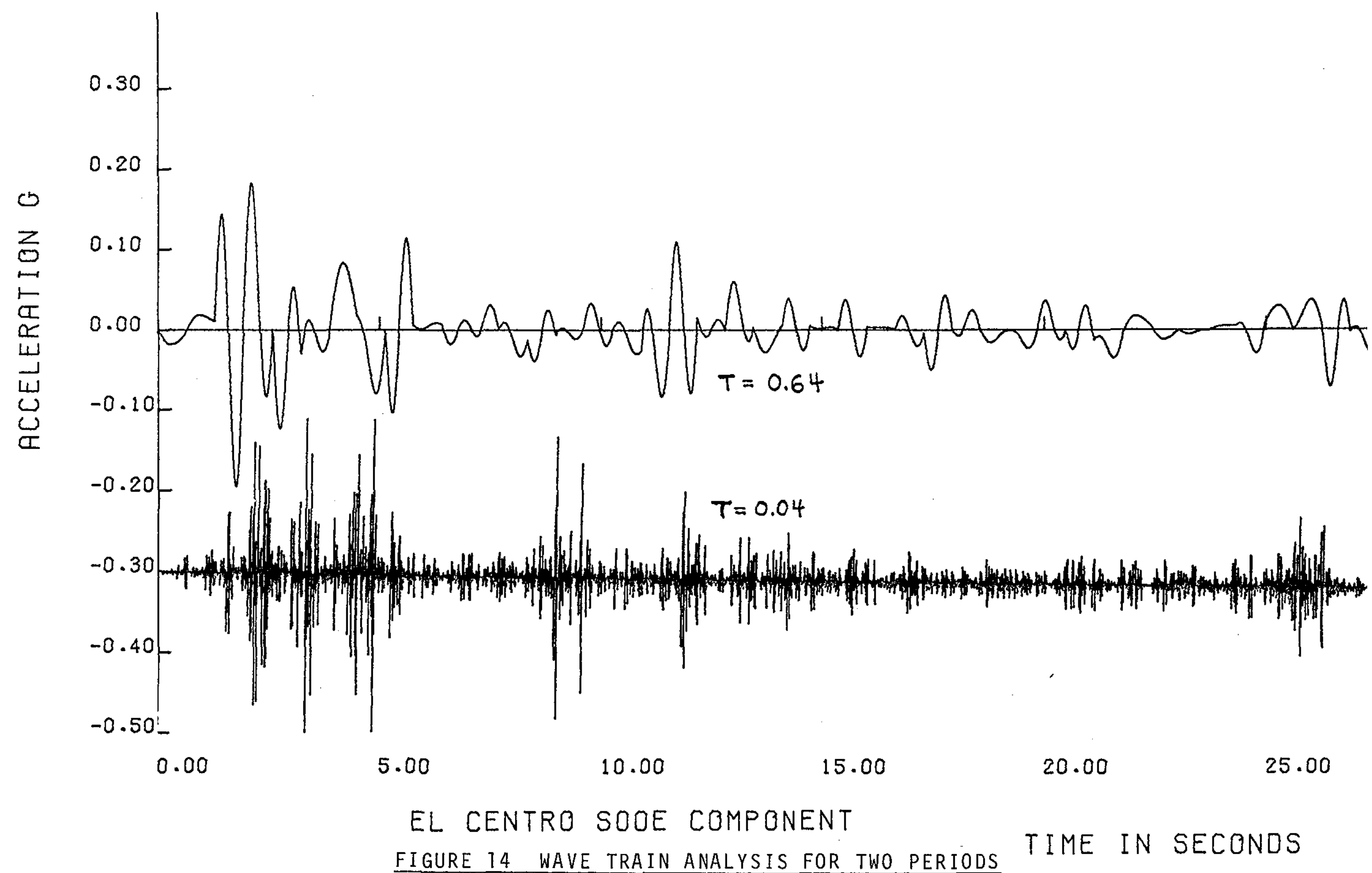


FIGURE 13 RMS OF PEAKS VERSUS MEAN OF PEAKS



EL CENTRO SOOE COMPONENT

FIGURE 14 WAVE TRAIN ANALYSIS FOR TWO PERIODS

A natural reluctance exists to making changes in a code such as ACI 318 based on technical improvement at the possible expense of increased complexity and without a promise of an improved competitive position of reinforced concrete compared to structural steel.

The possibility also exists that these changes will influence the economic position of one segment of the concrete construction industry compared to another. This situation arises in comparing the reliability of cast-in-place continuous construction with precast industrialized construction. The present code does not differentiate between the two types of construction, although there is ample evidence to show that continuity has the influence of increasing reliability. If safety to building occupants under rare extreme loads is of interest, cast-in-place continuous construction has a large advantage over industrial construction.

The issue can be readily appreciated by comparing the bending failure of a precast simply-supported slab floor composed of separate elements with one having the same nominal strength but cast-in-place and continuous with the supporting walls. The precast slab has a mean strength of m and a coefficient of variation of strength of V. If the probability distribution (PDF) of strength is as shown in Figure 15a, the probability of failure under a given loading, w, is equal to the indicated area of the PDF, and the reliability of the slab is equal to the balance of the area.

Assume that the continuous slab has equal amounts of midspan and support steel with independent moment capacities, means of 0.5m and the same coefficients of variation, V, as with the precast slab. The mean strength of the precast and the continuous slabs are identical. However, the standard deviation of capacity of the continuous span is 0.61 of that of the precast slab, so that the PDF of capacity is as shown in Figure 15b. The net result is a much smaller probability of failure for the selected load level compared to that with the precast slab.

This example was chosen to illustrate the influence of continuity in a very simple example. The issue is a change in reliability through the difference between the reliability of a continuous assembly and an assembly of separate components.

The other important consideration is that of balanced reliability. That is, a structure can be expected to experience many different loadings, each of which may involve different combinations of components. It appears desirable to have such a balance of reliabilities among the various systems and the forecast loads, so that the overall losses during the life of the facility are a minimum. Such analyses are only possible with thorough probabilistic methodology.

DECISION MAKING

The loading that will occur during the life of the structure is uncertain and can only be described in probabilistic terms. Thus, for any given life of structure, the result of a scientific investigation is a set of load levels with associated probabilities that these levels will prove to be maxi-

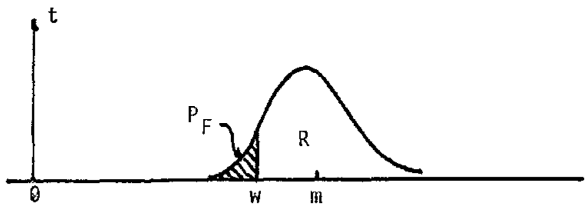


Figure 15a Precast

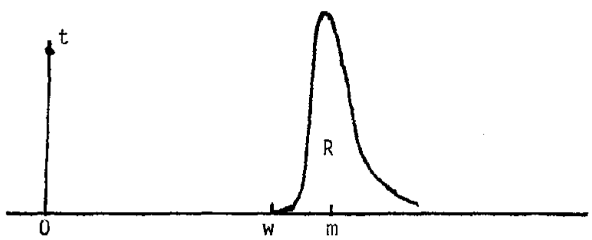


Figure 15b Continuous

FIGURE 15 COMPARISON OF RELIABILITY OF PRECAST AND CONTINUOUS CONSTRUCTION mum values. A design cannot be made based on such input for there is no way to design a probabilistic member to support an uncertain loading.

The two missing ingredients are the set of design options or alternatives and the costs-benefits associated with each possible design coupled with each possible loading and its likelihood of occurrence. From the standpoint of a specific design, such calculations can be made and the optimum design determined. Codes, however, must include such a wide range of possibilities that criteria decisions are matters of judgment in which the scientific forecasts are only a small part of the problem and the decisions are almost entirely judgmental.

CONCLUSION

The complexity of the issues involved in assessment of seismic criteria using probabilistic methods has been discussed including problem areas as well as the state-of-the-art. Problems associated with geology and tectonics have not been included owing to the fact that such data are not presented in a rational probabilistic format, but rather rely strongly on subjective judgment and unquantified conservatism.

The development of applied probabilistic methods has been extremely rapid in the past. The rush to standardization has slowed this development, but this is in part a pragmatic recognition of the value of the tool itself.

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THE PURPOSE AND EFFECTS OF EARTHQUAKE CODES: A CASE STUDY OF SEMIPROBABILISTIC APPROACH

by

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I. PURPOSE OF EARTHQUAKE CODES

Earthquake codes are legally enforceable rules for the design and construction of new buildings and for the rehabilitation of existing older structures. These rules are intended to satisfy two basic objectives:

- a substantial and economically acceptable degree of protection against injury and property damage due to the effects of the moderate earthquakes (ranging from 5 to 7 on the Richter scale) which may be expected to occur during the economic life of a structure;

- an acceptable assurance that lives will be protected and structural collapse prevented under the effects of a large, catastrophic earthquake which might possible (though quite improbably) occur during the structure life.

It should be recognized that some risk must always be accepted, since earthquakes are future random events -- and for every earthquake that has occurred, there may be a bigger one coming. The objective is to reduce the chance of damage or injury due to quakes down to the acceptable risks that we accept during the course of normal life.

A most essential requirement of these code design rules is that they be applicable to all construction, such that the entire public is protected. The rules must hage this quality of universal applicability if future disasters are to be avoided.

I.1 The Relation of Design Loads and Quality of Structure

Engineers model the effects of strong earthquakes as an application of lateral forces on a building. These represent the inertial loads as the structure is accelerated from side to side during the quake. The phenomenon may be visualized as the push of a giant hand on the side of a building. The results of this "push" are a sidewise bending or drift of the structure, and intensified forces, moments, and shears in the columns, beams, and walls of the building (see Fig. 1).

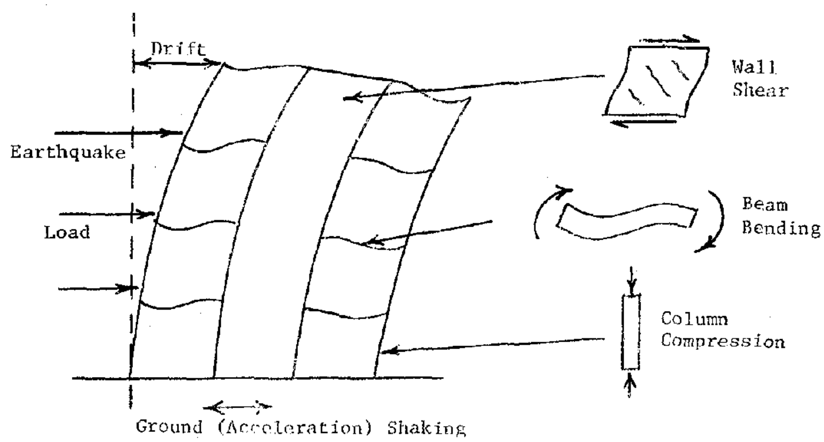


Figure 1.

The occurrence of a strong earthquake may cause drift values of such magnitude that windows, interior walls, elevators, and service ducts are seriously damaged; therefore, the control of this drift by means of stiffening braces and walls is a most essential engineering objective. Further, the individual structural elements must be both strong and tough enough such that:

columns do not buckle and collapse

Fig. 2

beams do not fracture

Fig. 3

and walls do not shear apart.

Fig. 4

It is most important to realize that an earthquake can have widely differing effects on different types of buildings, depending upon their qualities of (a) symmetry and regularity and (b) non-symmetry and non-regularity.

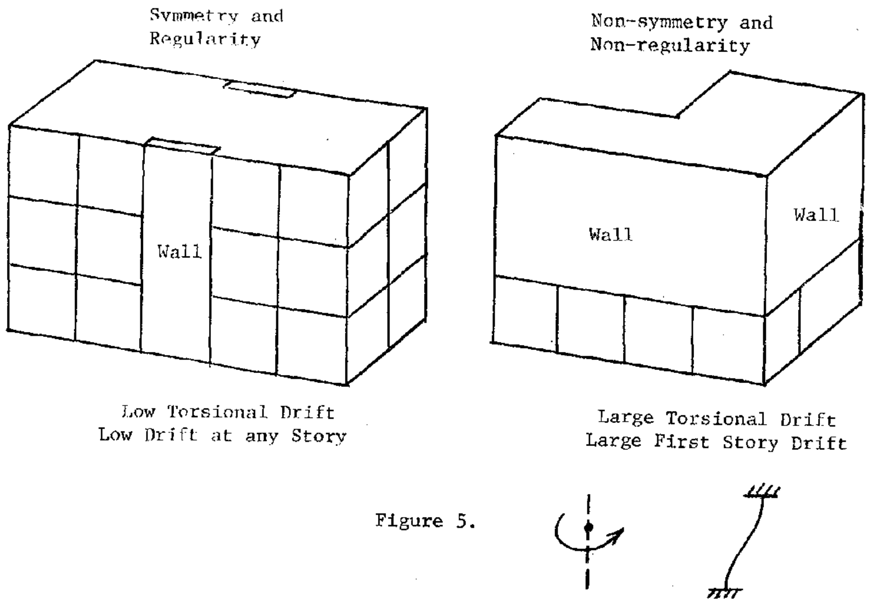


Figure 5.

If a building is well-braced by walls, regular and symmetrical, drift is easily controlled. If, however, there are drastic irregularities from floor to floor, or if the plan is grossly non-symmetrical in its floor plan or with walls on one side and flexible framing on the other -- then severe localized drift and torsional twisting distortions will occur.

If the members are both strong and tough, then the shaking punishment of the quake can be absorbed. If the members are brittle, due to poor materials, careless design and construction, or if the connections of the walls, floors, and frame are weak, then only very moderate earthquakes can create damage and collapse.

Bearing all of these building qualities in mind, engineers employ design loads (the representation of the side push of the earthquake) as a means of proportioning the strength or size of the beams, columns, and walls of a building. If a structure is regular, symmetrical, and possesses tough and well-constructed elements, then it has natural qualities of earthquake drift control and energy absorption. The design loads for this structure can be set at a relativ ely low level without endangering the quake-resistant capabilities. On the other hand, if there are irregularities in configuration or briggleness in the construction, then these weaknesses must be compensated for by large design loads with their resulting large member sizes -- this is necessary in order to provide the same quake resistance as the regular tough structure with its low design loads. From the economic standpoint, large design loads mean large material and construction costs, and low design loads therefore have a distinct cost advantage, providing that the proposed structure can qualify for these low loads.

I.2 History of Earthquake-Loading Criteria

In the 1930's, 1940's, and 1950's the structural engineers of California generated the basic earthquake code and design procedures which are widely employed throughout the world today. It is most important to recognize that these engineers had developed these provisions for the types of building construction which were prevalent in California at that time -- specifically structures in Los Angeles and San Francisco. These buildings typically had strong steel or reinforced concrete framing skeletons, filled in with very well-constructed brick masonry walls and strong concrete flooring systems. They were usually symmetrical and regular in their configuration, and in most cases they qualified as good, tough, earthquake-resistant structures. It is a most educational experience to walk along Market Street in San Francisco and see some of these structures that survived the motion effects of the disastrous 1906 Earthquake without even significant damage. The California engineers, having a knowledge of the good performance record of these structures, formulated the following type of design philosophy:

- relatively low lateral earthquake forces for the design of structural members,
- relatively strict rules governing the types of allowable materials, the methods of member design and tough connections, and an implied need for symmetry and regularity.

For the time up to the 1960's, before which much construction in California did not differ substantially from that of the tough buildings, this philosophy was appropriate to provide seismic resistant structure. However, architectural configurations along with methods of construction have changed significantly in the past two decades. Frames have become much more open and irregular, and the rugged systems of masonry partition walls and concrete floors have been replaced by largely pre-fabricated elements with very flexible characteristics. The low seismic design forces which were quite appropriate for the classical old methods of construction were applied without change for the newer structures. Moreover, the "California" type code and its low design values were adopted for new buildings throughout Central and South America. The basic error was that the new buildings did not have the regularity, stiffness, and reserve toughness necessary to justify the classical low design values.

The Caracas, Venezuela, earthquake of 1967 showed that reinforced concrete framing had to be made much more tough or ductile in order to survive even moderate earthquake effects without collapse.

The San Fernando, California, quake of 1971 provided a similar lesson for the correction of brittle concrete frame behavior.

The Managua quake of 1972 re-emphasized the experiences and lessons of the previous events, and added the concept that a large amount of damage can be caused in buildings where the framing is too flexible and excessive drift occurs. Also, if buildings are not symmetrical, then the engineer must consider the effects of twisting or torsional drift.

Thus the engineering profession learned from these experiences, and they worked within their committees to upgrade existing codes as follows:

- load levels were to be increased moderately, based on the seismic region and local conditions,
- concrete framing was to be ductile,
- limits were set on drift, such that frames had to be made stiffer or more walls and bracing were required,
- the engineering analysis should consider the effects of building irregularity and non-symmetry.

However, while the engineers were doing their necessarily methodical improvements, there was a public demand for more immediate action.

I.3 Reaction and Response after a Major Destructive Earthquake

In the chaos of rescue, public care, debris removal, and fire control which is the usual result of earthquakes in large cities, very few individuals are concerned about why some buildings survive and other collapse. The attention is focused on the collapsed structures, and the reaction is that these failures should never be allowed to happen again. The public, through their representative officials ask, "Why the collapse?" Engineers state that they have used the design loads from the California Codes. Officials call for doubling or tripling of these loads (based on the advice of theoreticians and seismic experts) to avoid future disasters, and herein begins the problem: more analysis of soil and structure, more complex design -- with higher costs and delayed reconstruction as a result. The sad part is that not only are all of these supposedly corrective emergency actions not particularly effective, but they are actually very detrimental on a social and economic plane in a region or country where new construction is essential for public needs.

The most effective action is not accomplished uniquely by design load magnification, but rather in the correction of bad building configurations and the elimination of the brittle construction that is prevalent in some tyeps of new construction and in older buildings. The basic lesson is that good buildings do not require high earthquake design load levels which would substantially affect their construction cost.

Another complicating factor occurs which further increases costs and delay. Public officials, in their search for the best answers for corrective action, frequently ignore (at least for awhile) the basic structural engineering viewpoint and turn to the academic and scientific community for advise. This latter group contains the seismologists, geologists, mathematical soil and structural analysts. They exist within the universities, governmental agencies, and consulting firms specializing in geotechnical work or advanced computer analyses. These experts all have very important areas of knowledge, and with all good intentions they wish to see that this knowledge is immediately utilized for public protection.

This academic, scientific, and advanced analytical work is certainly important and does eventually contribute to code improvements -- but the implementation of effective codes should not be delayed nor should their contents be overly complicated by the required inclusion of advanced methodologies. These include the following specialty areas:

- Site exploration. The research of all available geological information and the physical trenching and exploration necessary to detect active earthquake faults or any other sources of hazards such as land slides or settlement.

- Site response analysis. The research of all available geological and seismological information necessary to predict descriptive characteristics of future earthquake motion at the bedrock level under the structure site. Soil exploration and drilling is then performed to determine the dynamic properties of the soil layers between the bedrock and the structure. A mathematical model of the soil layers is formulated and computer analyses are performed to predict the surface response of the earthquake motions at the bedrock level. These calculated surface motions are then employed for the design of the structure.

- Advanced dynamic response analysis. Given either past earthquake records or the results of the site response analysis, computer analysis provides a complete record of the seismic response of a proposed building design. Engineers employ the results to verify the design strength and drift control of the structure.

Again, these are all valid areas of investigation and analysis, and it is definitely not intended here to say that the work in these areas is not necessary. The outlined operations are necessary for important and unusual structures and for special site conditions, but good judgment is required in the definition of criteria before they are made to apply to general classes of ordinary structures.

In the review of the effects of recent earthquakes, no building failures have been due uniquely to the absence of knowledge that would have been provided by the above-listed specialty investigations and analyses, other than perhaps risk zoning for the appropriate design load levels based on the seismicity of a region. Practically all of the past failures would not have occurred if the building design had conformed to the letter and intent of the 1973 (and even more definitely by the 1976) Uniform Building Code. This is to say that good codes now exist to provide appropriate load levels and corresponding methods of design for given building types and materials.

Therefore, future code development work does not require inclusion of more refined analyses of seismicity, site response, or building response, but rather it involves a clarification or educational process that will allow all designers of buildings to understand the intent of our most current codes, and to understand the way in which a properly designed building resists drift damage effects during a moderate earthquake and provides against collapse during a catastropic earthquake. The present code load levels and methods of design are sufficient, but the present code does lack the element of rationality. With this weakness, it is easy for inexperienced designers to misinterpret the provisions of the code and thereby create unsafe structures. It is therefore most necessary to rewrite the format of the code so as to properly define appropriate design force levels for the various types and configurations of structures, and to provide a rational relationship of each design step to the actual earthquake response.

A good "rational" code should make every designer aware of the following concepts and procedures:

(1) At the site where his structure is to be located, there can orrur two important levels of earthquake ground motions:

- a moderate earthquake for which damage must be controlled,
- a major or catastrophic earthquake for which collapse must be prevented.

A zone map should furnish the design load information representing these two earthquakes -- each having its own acceptable risks of occurrence at the site of the structure and consistent with its degree of importance.

(2) An analysis procedure should be defined which predicts the forces and deformations of the moderate earthquake and the deformations of the major earchquake on the proposed building structure. This should include methods of predicting the seismic behavior of non-symmetrical or non-regular structures. Specifically, there should be distinct definitions of the sites, structures, materials, and occupancies for which the simplified code procedures are applicable, along with the distinct definitions of sites, structures, materials and occupancies where more refined methods are required. These methods include the areas of geological investigations, site response studies, and the various levels of dynamic analysis. Engineers must know the limitations of the necessarily simplified assumptions and methodologies of the code. They msut know when the uniqueness, complexity, or importance of a structure requires special studies, analyses, and design methods beyond those of the code.

(3) Rules should be given that identify the earthquake design loads appropriate for the given building properties of:

- earthquake force-resisting system,
- Building configuration,
- type of materials,
- type of member design and their connections,
- quality of construction,
- quality of supervision

(4) Rules should be given for member design and structural system configurations such that the members, connections, and the total system can provide the toughness and stability for damage control and collapse prevention.

(5) Structural and non-structural damage control should be verified at the design load and the deformations of the moderate earthquake.

(6) Structural stability and collapse prevention should be verified at the deformations of the major earthquake.

(7) There should be proper definition of the building plans and specifications, and enforcement should be provided during construction such that the as-built structure conforms to the design.

I.4 The Necessary Objectives and Qualities of a Workable Seismic Code

Basically, a code is merely a set of rules which must create a building structure that is capable of providing a desired level of safety or protection against a given seismic risk. In the interest of or in the real necessity of simplification, it does not matter how accurately the rules resemble actual earthquake motions or the structural response to these motions. The only fundamental requirement is that the desired protection is provided within the economic constraints of limited engineering and construction budgets.

The expressed desires of the structural engineering community are, in order of priority:

(1) Simplicity, such that all provisions are well understood without misinterpretation, and all design operations can be accomplished within the established fee-structure for engineering services or design budget.

(2) Rationality -- each load specification, analysis procedure, and structure resistance provision should have a direct relationship to seismicity, ground motion, structure response, and structural element behavior due to structure response.

(3) Freedom to use responsible ingenuity for special structures -- there must be the opportunity for capable engineers to apply their perticular expertise in specification of seismic ground motion, methods of analysis, and formulation of the structural resistance system for special structures beyond the scope of the code. Along with this freedom, there should be definite responsibility for final design results.

(4) Reward and encouragement for dynamic and computer analyses, when merited either by the complexity of a given structure and/or the particular description of the input time history or response spectrum at a given site. Some reasonable restrictions are of course necessary to make sure that results are not unjustifiably different from the code base shear method for the case of regular structures. This is to prevent the practice of manipulation for the purposes of avoiding reasonable code provisions for regular structures.

I.5 Tough Connections and Drift Control Rather than High Design Forces

The widely publicized effects of destructive earthquakes cause a continual demand for structural engineers to raise the level of code design forces. While the objective of this demand is to provide safe structures free from collapse, the result is not particularly successful. The 1976 Uniform Building Code provides a good example: as a result of the political pressures for safety after the 1971 San Fernando Earthquake, code design loads were nearly doubled from those given in the 1973 UBC. However, after nearly a year of availability, the 1976 UBC has not found any degree of enthusiastic acceptance. It appears that the previous (1973) UBC gave design values which were just about optimum as far as economical design and construction are concerned. The 1976 UBC produces requirements for member section sizes, wall over-turning moment resistance, and wall shear resistance which have not been in evidence as the real cause of structural failures during destructive earthquakes, and hence this code does not inspire credibility.

It is predicted that the future trend of design codes will be as follows:

- design loads will not be too different from the 1973 UBC, even in regions of high seismicity;
- safety from damage and collapse will be achieved by
  - (a) strict specifications for tough, damage-resistant connections and structural systems,
  - (b) drift or deformation limits for control of non-structural damage.

In summary, good design details will be emphasized rather than the high load levels that have been given more by theoretical analysis than by any observed behavior during actual major earthquakes.

I.6 The Trend of New Structural Systems

This section is to provide a discussion of existing and future economic conditions concerning the building construction industry and the functionality of buildings as required by their occupants. Topics to be discussed are:

- structural systems treated in existing codes,
- cost, availability and effective utilization of skilled construction workers,
- current and future building systems,
- recognition in a new code.

Structural systems treated in existing codes -- In the past few years there has been great concentration by code-writing committees on the creation of ductile steel frames and on ductile, reinforced-concrete frames and shear walls. These provisions are essential to assure safe performance at reasonable design load levels. However, a basic fact must be recognized: construction costs are rising to such an extent that building developers are turning to other, cheaper methods of construction. There appears to be a definite decrease in poured-in-place concrete frame structures. Also, when frames are used, modern demands are for long spans and clear floor areas -- such that prestressed elements are necessary for both economic and story height clearance reasons. The question is, "Have we created a dinosaur (ductile frames) which may become extinct?" If it is reqlly required for structural safety, then strong pressure from engineering groups is required to overcome economic demands.

Cost, availability and utilization of skilled construction workers -- The cost of labor for skilled workmen is increasing, and their availability is decreasing. Hence, building developers are turning toward the use of pre-cast procedures of construction or other forms that decrease the need of skilled workmen in the field.

Current and future building systems --

- Slip-form, poured-in-place walls with precast structural frame and floors. Beams are usually prestressed concrete.
- Multi-story masonry (hollow-block) walls with pre-cast floors.
- Prestressed, post-tensioned rigid frames, with precast floor systems, or with post-tensioned slabs.
- Pre-cast shear wall panels post-tensioned to pre-cast floor systems.
- Tilt-up wall industrial structures.

Recognition in a new code -- While structural engineers might prefer the regular, symmetrical, ductile frame and shear wall structures, the pressures of economics and the requirements of functionality by building owners often dictate that other structures be built. An effective new code must recognize this situation and provide for rules of analysis and design for structures which are not particularly ideal for effective seismic resistance and therefore need careful design and supervision. Existing codes, such as the 1973 and 1976 UBC, have been restrictive in the high seismic zones of California (such as San Francisco and Los Angeles) with respect to allowable lateral force systems (such as the requirement for ductile, reinforced-concrete provisions for seismic frames). It is possible that the relatively less severe seismicity of Costa Rica would permit a wider scope of allowable systems (such as semi-ductile concrete frames with or without prestressing) so long as the systems are properly classified in the code with respect to appropriate design load levels, allowable configurations, and detailing requirements for members and connections.

II. INTRODUCTION TO THE PROPOSED SEISMIC DESIGN PROCEDURE

II.1

In this section a general overview of the seismic design methodology developed through this research is presented. A short description of all major parameters and steps is given to provide the reader with a quick comprehension. This section can be viewed as a summary of the work that follows in detail in succeeding sections.

This portion of the report is dedicated to the goal of correcting an inherent flaw in the attitude of the typical structural engineer. Specifically, an engineer will devote many years of his life to mastering the art of structural analysis, both during his school years and in his rare leisure hours. He will dutifully learn the classic methods of indeterminate structures and then gleefully branch out to the fascinating fields of matrix methods and computer analyses. However, a very strange fact is that this same dedicated individual will accept -- without the slightest question or doubt -- a very simplified version, such as $V = ZKCW$, for a very complicated and interesting phenomenon known as earthquake loading. It is perhaps due to the engineering education, but somehow engineers are always prone to over-analyze a structure for loads which are at best "crude" and often rather "inappropriate" for the structural environment. Therefore, this report will attempt to establish the principle that a reasonable fraction of the engineering intellect should be devoted to load analysis, along with the inevitable preoccupation with structural analysis. The reader, therefore, should prepare himself for an onslaught of statistical response spectrum analysis, probabilistic description of uncertainty, damping and damage excursions, and some questions of structure configuration, material behavior, and construction quality. It is hoped that he or she will be a better engineer, and that buildings will be more reliable because of this effort.

In order to design economical buildings which will perform adequately during strong earthquake ground motions, it is necessary for structural engineers to have a practical understanding of:

- the probability of occurrance of important levels of earthquakes,
- the acceptable risk associated with these events for different use classes of structures,
- the representation of earthquakes in terms of response spectra at the structure site,
- the earthquake demands on the strength, stiffness, ductivility, and energy dissipation capacity of various structural systems,
- the design of the structural elements and lateral force-resisting system such that the important levels of earthquakes may be resisted with acceptable reliabilities of performance.

In the paragraphs which follow, a seismic design procedure is formulated which hopefully will provide the engineer with this needed understanding. In order to assist the reader in the organization of the presented material, the following general description of the design method is given.

II.2 Design Objectives

For a given lifetime of a structure, an adequate design should provide acceptable reliabilities of protection against:

- excessive damage due to a moderate or damage-threshold earthquake,
- condemnation due to a major or condemnation threshold earthquake,
- collapse due to a catastrophic earthquake.

The value of the acceptable reliabilities of protection against each level of earthquake depends on the use class or importance of the structure. The concept of cost of protection versus seismic risk should be considered in this evaluation.

Moderate, major, and catastrophic earthquakes are described in terms of the seismicity at the structure site. This seismicity is expressed in terms of probabilities of peak ground accelerations for a given time period, and also in terms of the corresponding response spectrum values.

Damage control and condemnation protection are accomplished through strength requirements and deformation limitations of the structure response to moderate and major earthquake response spectra. This requires a classification of structural systems according to their respective deformation capacity at the damage threshold and ductility at the condemnation threshold.

Collapse protection against a catastrophic event is maintained by specific restrictions on the types of allowable lateral force resisting systems. These systems must all have the characteristics of maintaining vertical load-carrying capability under severe lateral deformations.

II.3 Methodology

To achieve the above design objectives, the following methodology is formulated:

(1) Forecasting of future seismic events. Develop occurrence rate of peak ground acceleration at site and site response spectra.

(2) Select peak ground acceleration and response spectra shapes for moderate (damage threshold) and major (condemnation threshold) earthquakes according to local site conditions, structure use class, and acceptable risk level.

(3) Develop structure design spectra for different types of structural systems according to deformation characteristics and reliability of the system.

(4) Develop procedures for computing the response of structures to the above design spectra (modal superposition or base shear method).

(5) Develop criteria for the design of structural systems and members (strength, ductility, drift, P-Delta effect).

All steps of the methodology and a detailed design procedure are discussed in detail in Shah et al. (1976) and Mortgat et al. (1977). Presented below are brief summaries of the most important elements of the procedure.

II.4 Site Response Spectra

For a given region with known (overall) geological characteristics, a sample set of past major earthquake accelerographs and their corresponding response spectra can be assembled. This data set may be from the region for which seismic design criteria are to be developed or from geologically similar regions. Each response spectrum is then scaled so as to have a unit value of peak ground acceleration (PGA), and is hence termed as a dynamic amplification factor (DAF). The resulting sample set of DAF's is then averaged to form the mean DAF (MDAF) which provides the representative spectral shape for the given region. This shape may be adjusted for known hard or soft soil column effects at the site. Given any forecasted PGA value for a future earthquake, the acceleration response spectrum may be obtained by multiplying the MDAF by the PGA value.

The spectrum as obtained from the basic data of instrument time history readings is then converted to an "effective" structure response spectrum by means of a reduction factor R, which is discussed in detail in Shah et al. (1976).

II.5 Peak Ground Acceleration

The PGA values at specific sites in any region which have a probability P of being exceeded during a given economic lifetime of a structure are presented in the Acceleration Zone Graphs or the Iso-acceleration Maps developed probabilistically and discussed in cited references. The PGA values for the damage threshold and condemnation threshold earthquakes are termed A_D and A_C, respectively.

A seismic event, X, having a probability of exceedence, P_X, is adequately described for design purposes by the PGA value from the Acceleration Zone Graph, A_X, and the regional spectral shape, MDAF.

II.6 Structure Use Class and Risk Levels

Planners are able to categorize the various structure uses into classes, depending on their importance and need before, during, and after a strong earthquake. Since it is neither practical nor economically feasible to provide a damage-resistant structure for all conceivable levels of earthquake ground motions, each use class will have to have assigned its own particular probability or risk of repairable damage, P_D, and corresponding risk of total condemnation, P_C, during its economic life. This risks should of course be very low for essential facilities such as hospitals, and may be relatively high for a purely functional structure such as a warehouse. The risk of total collapse can be virtually eliminated by code restrictions on he type and quality of the lateral force-resisting system in a building.

The importance of the assigned acceptable risk values of P_D and P_C for each structure use class is that they, along with the site location, determine the corresponding values of A_D and A_C from the Acceleration Zone Graphs or the Iso-acceleration Maps.

The design objectives are then to assure a reliable level of damage control for earthquake levels up to a PGA of D_D, and condemnation prevention against the effects of an earthquake with a PGA of A_C. The A_D and A_C values are used to scale the mean response spectrum shape (MDAF) for design purposes.

II.7 Types of Structural Systems

The lateral force-resisting system may consist of rigid frames, bracing, and shear walls -- either in combination or in pure frame or wall systems (such as the K-Factor Systems of the UBC). Any permissible system must have the quality of collapse prevention; the vertical load-carrying system must remain intact under those catastrophic ground motions which may reasonably exceed the acceptable condemnation level.

Each structural system has its own characteristics of response to the the damage and condemnation threshold earthquake loadings. The measures used to avaluate these thresholds are: extent of repairable damage, ductility and energy-dissipation characteristics, redundancy of the system, quality control and degree of construction supervision, and reliability of performance in past earthquakes. Also, each particular system has its own value of total damping as it relates to the site response spectrum.

II.8 Structure Design Spectra

Given the structure site and use class, the risks P_D and P_C are known and the values A_D and A_C are found. Having selected the structural system type with its damping value, its reputation or reliability measure, and its ability to deform beyond its strength design level to a damage state and then further to a condemnation state, three design spectra are formed:

(1) Design Force Spectrum (DFS). This is an appropriately modified form of the spectrum for the acceptable damage threshold earthquake with PGA level A_D. The force response from this spectrum is used as the seismic design loading for the ultimate strength design of the structural members.

(2) Damage Deformation Spectrum (DDS). This provides the structure deformation demand of the earthquake with PGA level A_D, i.e., for the damage threshold event. The resulting deformations are used for computation of P-Delta effects, and for non-structural damage analyses (drift limitations).

(3) Condemnation Deformation Spectrum (CDS). This is the spectrum of the acceptable condemnation threshold earthquake with PGA level A_C. The resulting structure deformation response is used to estimate local member ductility demands and hence provides an approximate test whether or not these demands are within allowable limits. P-Delta effects and structural stability may be analyzed with these deformations.

Clearly, the most important of these three is the Design Force Spectrum (DFS), since its resulting design load levels must create a complete structural system such that the structural deformation response of the earthquake with PGA level A_D and risk P_D will remain reliably below the structure damage threshold. Also, in a structure designed for the DFS forces, the deformations of the earthquake with PGA level A_C and risk P_C will remain in most practical cases reliably below the structure-condemnation threshold. This spectrum must also meet the practical restrictions of economically feasible design, and in so doing it must not differ radically from the seismic load recommendations of modern codes. For overturning moment, a special spectrum termed Design Overturning Moment Spectrum (DMS) is developed for systems with ductile shielding of the vertical load-carrying members.

II.9 Computation of Response

The basic method chosen for the computation of the structural response is the modal superposition method. The use of this principle of superposition makes it necessary to employ a linear elastic model of the structure. However, this facilitates the computational effort in design offices, since computer programs for linear elastic response of two- and three-dimensional structural configurations are readily available.

Natural frequencies and mode shapes can be computed based on the mass distribution and deformation characteristics of the lateral force-resisting system, but should also include the effects of stiff elements that are not part of the lateral force-resisting system. Then, for a given spectrum (any one of the three design spectra), the structure response (force or deformation) is computed as the square root of the sum of the squares of the individual modal responses to the given spectrum (SRSS response).

For the case where the computed deformations are beyond the linear elastic range of the structure, it is assumed that the deformation response in the actual non-elastic structure is given by the SRSS deformation response of the lienar elastic model. It is recognized that this linear procedure can result in a certain amount of approximation error. However, this will be compensated for by an appropriate spectral confidence level and a requirement for special analysis for irregular structures.

For structures which meet certain requirements for regularity and symmetry, a simplified "base shear" method can be formulated. Empirical relations for structure periods, a base shear coefficient, and lateral force distribution will be given to provide a safe upper bound of design in lieu of the more lengthy modal analysis and response spectrum method. This is a most essential step in order to assure widespread application. However, even this simplified method will contain a descriptive commentary so that the designer is aware of the essential elements: earthquake levels and their associated risks; dynamic response of structures to these earthquakes; and design provisions for adequate behavior at the damage and condemnation thresholds.

II.10 Design Criteria

The seismic loads resulting from the Design Force Spectrum (DFS) response, together with ambient dead and live loads, determine the required ultimate strength capacity for member design. The ultimate strength design method based on elastic behavior of the structure is recommended for all types of structures, including steel structures. Load factors are suggested where deemed necessary.

Drift limitations are specified for the deformation response due to the Damage Deformation Spectrum (DDS), while secondary effects and structural stability are to be investigated at the Damage and Condemnation Deformation levels.

The ductility demand resulting from the Condemnation Deformation Spectrum response may affect the choice of the structural system and the detailing requirements for various elements such as boundary elements in shear walls and spandrel beams. In some cases, the CDS analysis may render it advisable to increase the strength of certain elements to keep the ductility demands below acceptable values.

II.11 The Role of Dynamic Analysis in Seismic Design

Dynamic analysis, either in response spectrum or time history form, has been prescribed by various recent seismic design recommendations and codes. This analysis may be an allowable alternative (or even a necessary requirement for special structures), as in the Uniform Building Code (1973, 1976). *However, nowhere in these seismic provisions is there given a definite and complete procedure of design based on a dynamic analysis. It is therefore the objective of this project to provide this very much needed complete procedure based on the response spectrum method. In addition to a more accurate determination of structure periods and lateral load distribution, the method allows the designer to have a direct physical and practical understanding of each step in the design procedure as it relates to seismicity and the related structural behavior. It is felt that this understanding is more important in a design procedure than the use of high design-load values, in order to create structures which can perform adequately during strong ground motion.*

II.12 Design Methodology

The design method is to be developed in terms of the following basic topics:

(1) Design objectives of damage control and condemnation prevention

(2) Seismicity in the form of is-acceleration maps and return periods

(3) Use classes of structures

(4) Types and behavior of structural systems

(5) Effective response spectra

(6) Design spectra

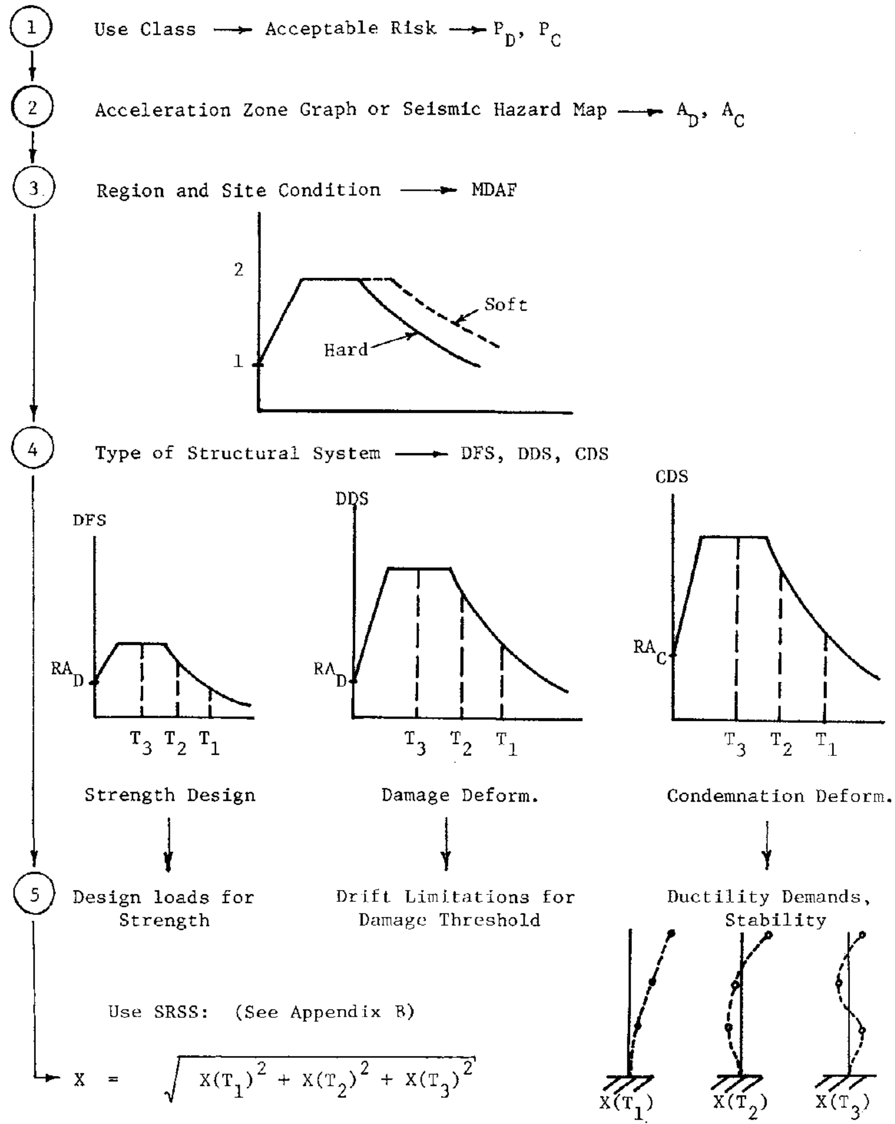
(7) Calculation of response

(8) Load combinations

(9) Member design

(10) Deformation analysis

A flow-chart representation of the design procedure is given in Figure 6.



FLOW CHART OF DESIGN PROCEDURE

Figure 6.

II.13 A Comparison of the 1976 SEAOC Recommendations and the Proposed Design Method

In order to best appreciate the proposed methodology, the following summary comparison is presented between the 1974 SEAOC recommendations (essentially equivalent to the 1976 Uniform Building Code) and the approach developed in this paper.

1974 SEAOC Recommendations -- The base shear for working stress design according to these recommendations is given by

$$V_B = AICSKW \tag{1}$$

where

V_B = Base shear to be distributed to each story according to a linear "empirical" version dynamic analysis.

Z = Seismic Zone Factor based on magnitudes of earthquakes in a region, but not on their frequency or chance of occurrence.

I = Structure Importance Factor. This value is greater than unity for essential facilities; however, it is not related to a definite acceptable value of risk.

C = an empirical shape factor for an inelastic multi-mode acceleration response spectrum. This is only a rough approximation of the statistical average of spectral shapes for the given region.

S = Site Response Factor for the influence of the underlying soil column and structure interaction on the spectral shape, as represented by C. It is a number larger than unity when the site period is near the structure period.

K = a reciprocal measure of the ductility of a given lateral force-resisting system. This value adjusts the inelastic response spectrum shape C so as to represent a reduction of lateral loads for ductile system and an increase for non-ductile system.

W = weight of the structure taken as dead load only -- with no ambient live load.

Within the actual design procedure, the following observations can be made.

- Ultimate Strength Design for Reinforced Concrete Frame Members is for factored dead load D plus live load L plus seismic effects E: $1.4(D + L + E)$, where E is the member load effect due to V_B. It is this particular method of load factoring that represents one of the principal differences between existing code (UBC) procedures and the proposed method. Traditionally, as explained in the history of codes, section I-2, code values for seismic load have tended to be lower (about 1/3 to 1/2) of the actual force requirements of a moderate or damage threshold earthquake. However, safety due to this actual motion is achieved indirectly by the particular method of load factoring, such as $1.4(D + L + \#)$, and by extra factors for shear wall shear stresses and for cross-bracing members. A numerical study of how the extra seismic-resisting capacity is achieved in spite of the low code E value by this indirect factoring is given in Shah et al. (1976). While adequate safety might be achieved by this code-factoring method, it is not very rational to design for a fairly heavily factored dead and full live load along with a rather low, non-realistic seismic load E. We should be much more concerned with adequate evaluation and factoring of the seismic E load rather than with the vertical dead and live loads, which are relatively well-known. Therefore, the proposed method prefers to use a reasonably conservative estimate of ambient vertical load (D = 0.4L) at the time of the quake plus the best available evaluation of seismic force requirements of the damage threshold ground motion (the DFS spectrum).

- There is no specific requirement for a verification of stability and condemnation protection at the major earthquake level (except for a special requirement for vertical load-carrying members at about four times working stress design deformation).

- There is no consideration of modal participation and effect of mode shapes on lateral load distribution (except for the top story force increment.

<u>Proposed Design Procedure</u> -- Base shear and lateral design load are given by the SRSS modal response to the Design Force Spectrum.

$$\mathrm{DFS} = R \cdot A_D \cdot (\mathrm{MDAF}) \frac{1}{d_T} (1 + k_T V_S) \qquad (2)$$

where

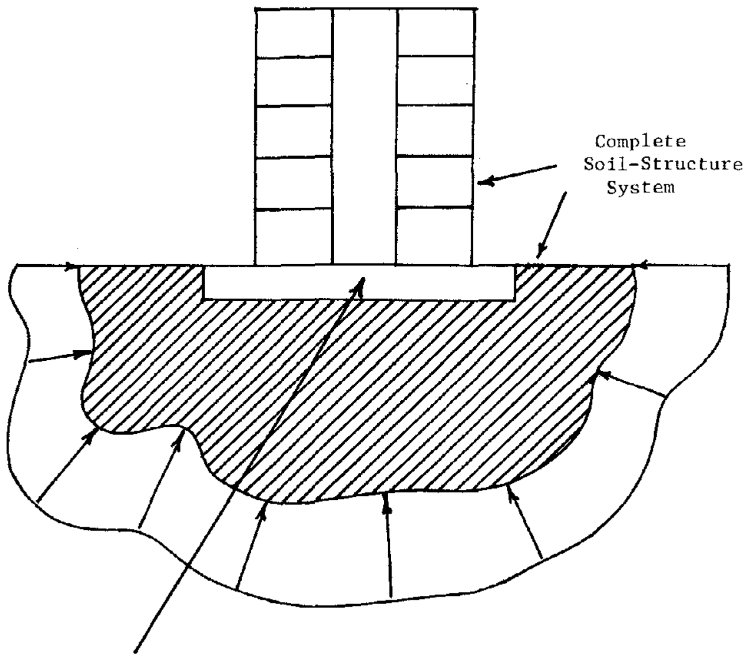
R = a peak acceleration reduction factor to represent the effective acceleration on the structure. It represents the spatial average of peak accelerations on the effective soil-structure system (see Figure 7).

A_D = peak ground acceleration at structure site -- having acceptable risk of being exceeded. If A_D is exceeded, then extensive structure damage may occur.

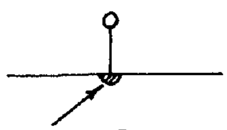
MDAF = mean or statistical average of acceleration response spectrum shapes for the region. The shape can include any soil-column response effects, and together with R can represent soil-structure interaction effects (see Figure 8).

d_T = damage defornation factor for a given lateral force-resisting system. It represents the ratio between the maximum acceptable deformation at the damage earthquake level and the design deformation in the highest stressed member. The d_T value depends on the K-factor type of the system (see Figure 9).

$(1 + k_T V_S)$ = spectral confidence interval factor, where V_S is the coefficient of variation of the spectral shape and k_T sets the confidence level. The factor k_T allows for the degree of reliability, inherent in a system, of attaining the given d_T distortion value without excessive damage. If a system is very reliable, then k_T may be zero (see Figure 10).

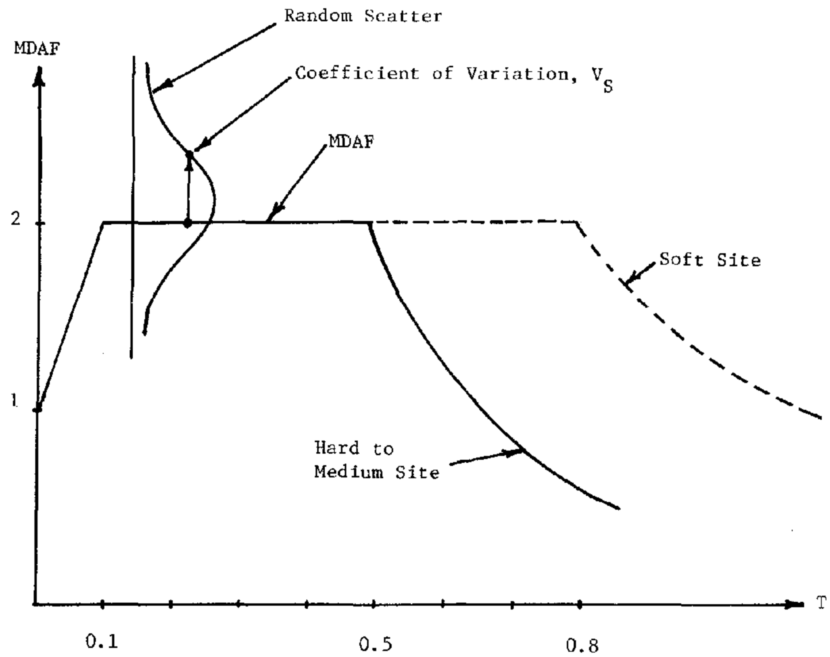


R · PGA = Surface Average of the Distributed Peaks at all points of the soil-structure system.



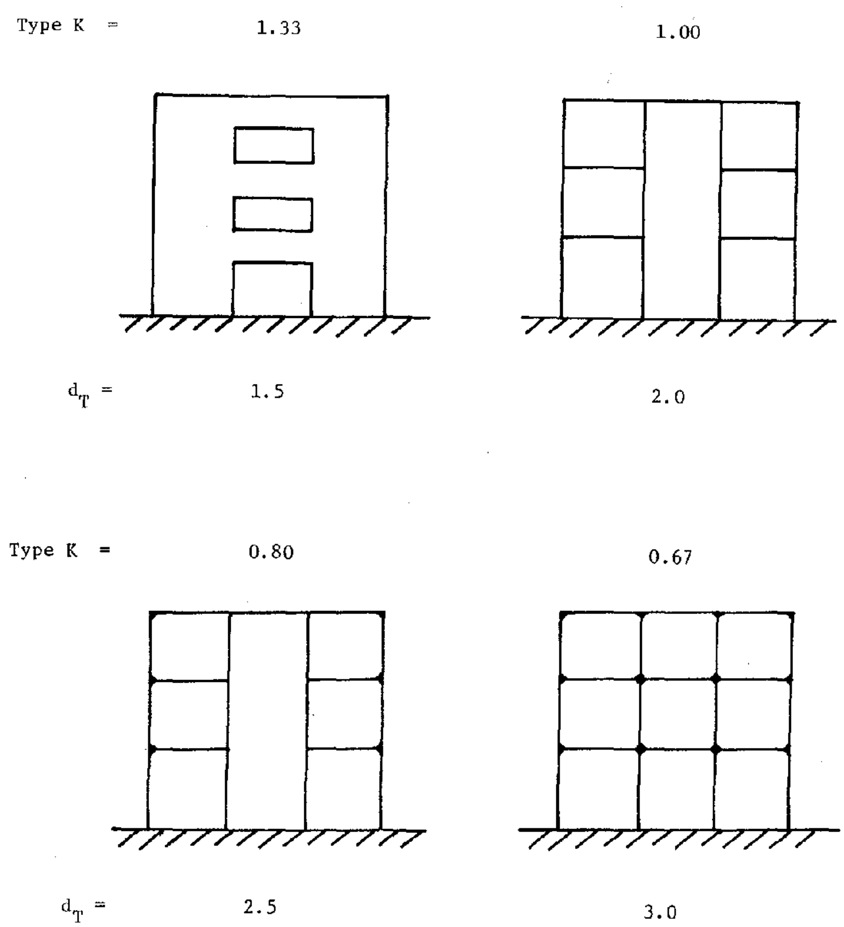
PICTORIAL REPRESENTATION OF R-FACTOR

Figure 7.



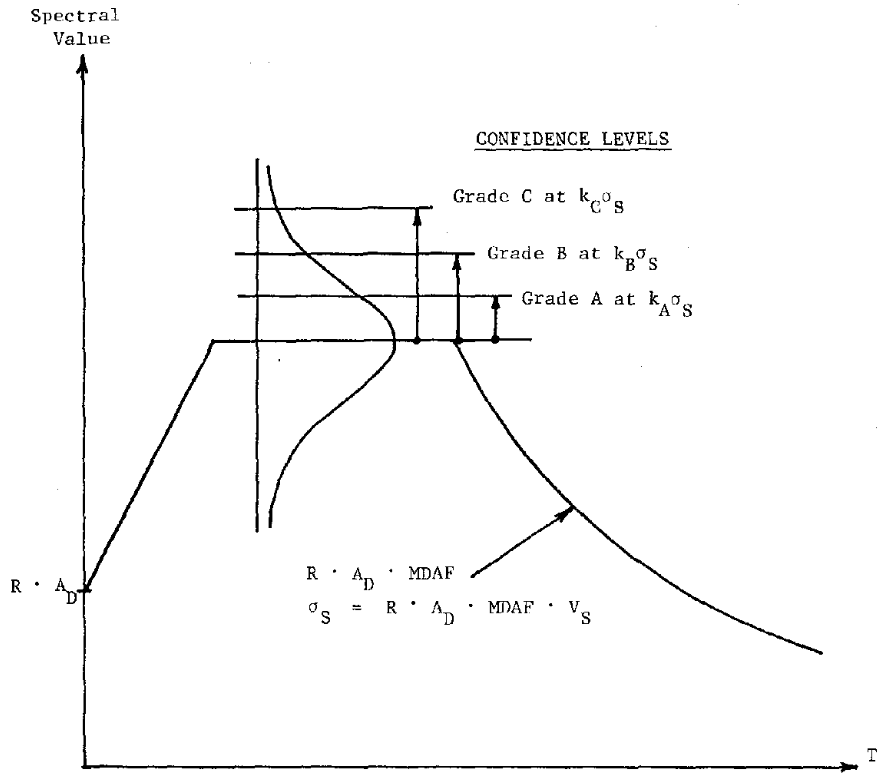
STATISTICAL PROPERTIES OF THE DAF SPECTRAL SHAPE

Figure 8.



d_T FOR THE VARIOUS TYPES
OF LATERAL FORCE RESISTING SYSTEMS

Figure 9.



RELATION OF GRADE TO DESIGN LEVEL

Figure 10.

$(1+k_TV_S)$ (cont.) The k_T value depends on the quality or grading of A, B, or C of a given structural system. See Figure 10 for the relation of confidence levels and the system grade of reliability.

Member seismic design forces are found by the SRSS value of the individual mode response to the DFS. In the formulation of the dynamic model, the full dead load and some reasonable fraction of the live load (0.4L) is considered.

Within this proposed approach, the following comments are pertinent.

- Strength design for members is the force response of the DFS plus dead load and a reasonable fraction of ambient live load (0.4L).

- Non-structural damage control is verified at the SRSS modal deformation response to the Damage Deformation Spectrum.

$$DDS = R \cdot A_D \cdot (MDAF)(1 + k_TV_S) = d_TDFS \tag{3}$$

  See Figure 11 for the relation of the linear model method of calculating SRSS response - to actual unknown random response to a given earthquake.

  This is a most important phase of the design procedure, since it requires the designer to consider the flexibility of the structure with respect to damage to the architectural, utility, and service facilities. These items represent a considerable portion of the structure value, and may be necessary for life safety.

- Local member ductility demand and structure stability verified at the SRSS modal deformation response to the Condemnation Deformation Spectrum,

$$CDS = R \cdot A_C \cdot (MDAF)(1 + k_TV_S) = \frac{A_C}{A_D} d_TDFS \tag{4}$$

  where A_C is the PGA value corresponding to the condemnation level seismic event (see Figure 11). Local member deformations are compared against their yield level deformations to assess whether ductility demands are within allowable limits.

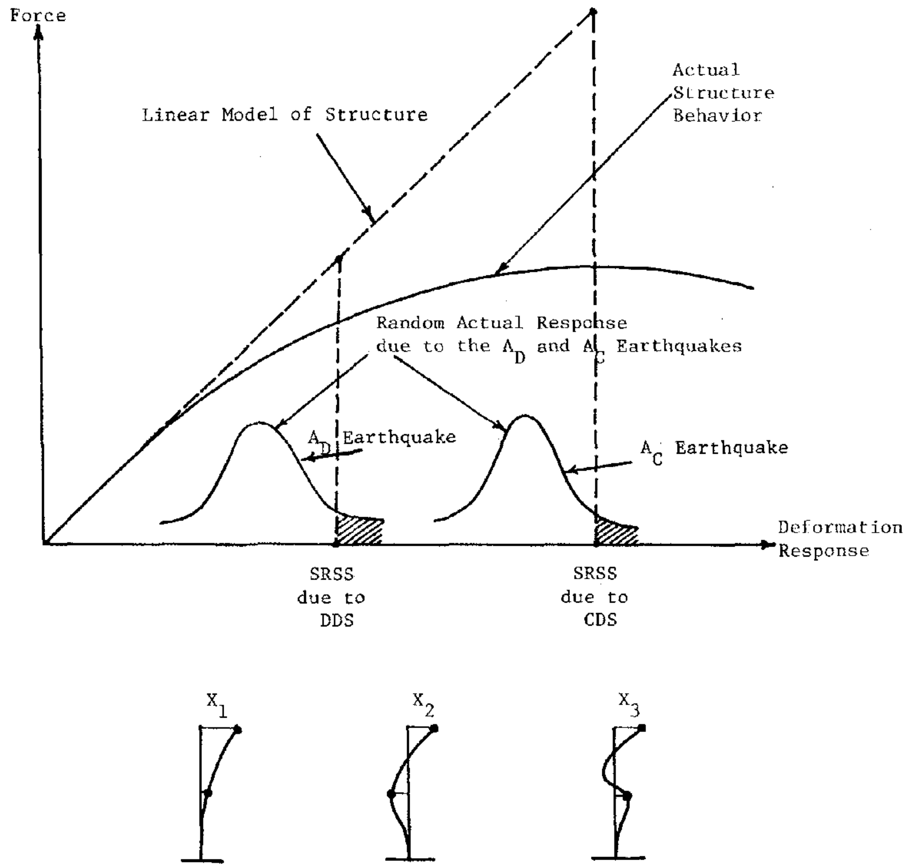
II.14 Basic Philosophy of the Proposed Seismic Design Procedure

In the design spectra, such as

$$DFS = R \cdot A(MDAF) \frac{1}{d_T} (1 + k_TV_S) \tag{2}$$
(repeated)

it should be noted that a very simplistic and approximate representation is given for some very complex phenomena. For example,

- R represents all soil-structure interaction effects.
- d_T and the β_T of the MDAF account for both damping and the nonlinear system effect of the "tuning out" of harmonic response.



$$SRSS = \sqrt{X_1^2 + X_2^2 + X_3^2}$$

RELATION OF LINEAR MODEL COMPUTED
RESPONSE TO ACTUAL STRUCTURE RESPONSE

Figure 11.

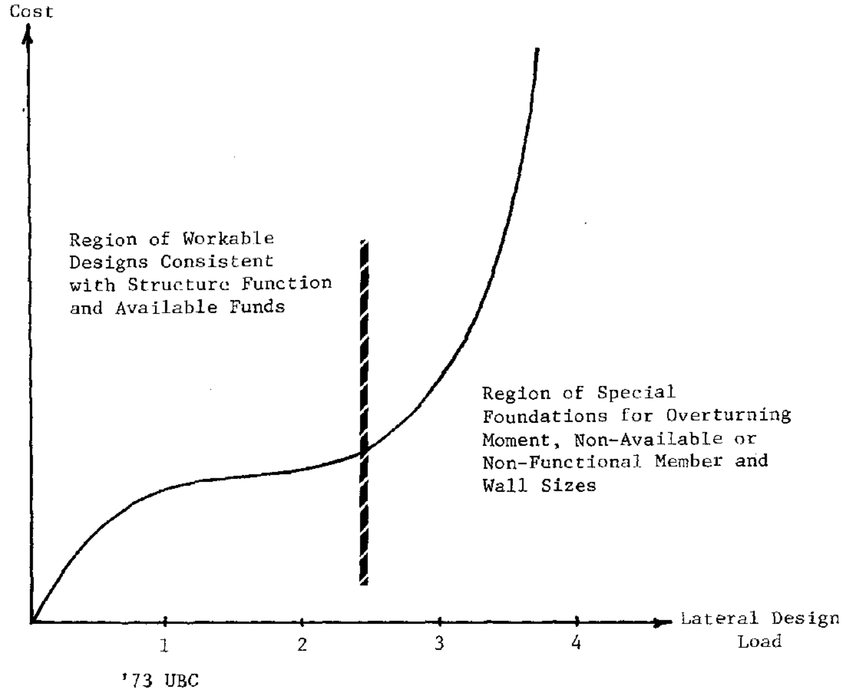
- The MDAF has two simple shapes, to allow for the soil column response effects.

Obviously a more complex representation of these and other structure response phenomena could have been proposed in order to better predict the effects of a future seismic event; the net result would be higher or lower design load levels, based on the specific structure and site conditions.

However, for this proposed design method, the following general philosophy has been adopted: given realistic seismic design load levels at the ultimate strength level, the accuracy in prediction of future seismic loads is not particularly necessary for the attainment of the design objectives of damage and condemnation prevention. The insensitivity to the cost of providing lateral load resistance within a certain range is illustrated in Figure 12. The principal element of the design philosophy is to provide procedures which will create a good seismic-resistant system having:

- at the damage threshold earthquake response,
  - adequate strength and stiffness for damage control,
- at the condemnation threshold earthquake,
  - no excess of inelastic deformations beyond the failure capacity of members and
  - no large imbalance of inelastic deformation in any story level of the elevation, or in any wall or frame line of the structure plan.

The proposed design procedure is based on this "good system" (rather than "precise load") philosophy and can attain the objectives by following the basic criteria of a response spectrum method.



ECONOMIC AND ENGINEERING CONSEQUENCES OF LATERAL LOAD CRITERIA
ASSUMING EQUAL DESIGN PROCEDURES AT MULTIPLES OF THE 1973 UBC

Figure 12.

III. A NUMERICAL EXAMPLE FOR MANAGUA, NICARAGUA, AND SAN JOSÉ, COSTA RICA

III.1

Tables 1 and 2 give the suggested damage and condemnation risk levels for Managua and the corresponding values of A_D and A_C. Similarly, Tables 3 and 4 give the values for San José, Costa Rica.

Table 1. Managua Region
Suggested Damage "Risk" Levels

Class	Economic life Yrs.	RP_D Yrs.	P_D	"Risk"/yr.	A_D g units
1	100	500	0.20	.002	.45
2	50	100	.40	.01	.35
3	20	50	.40	.02	.30

Table 2. Managua Region
Suggested Condemnation "Risk" Levels

Class	Economic life Yrs.	RP_C	P_C	"Risk"/yr.	A_C g units
1	100	1000	.1	.001	.47
2	50	500	.1	.002	.45
3	20	100	.2	.01	.35

Table 3. San José, Costa Rica Region
Suggested Damage "Risk" Levels

Class	Economic life Yrs.	RP_D Yrs.	P_D	"Risk"/yr.	A_D g units
1	100	500	.20	.002	0.31
2	50	100	.40	.01	0.18
3	20	50	.40	02	0.15

Table 4. San José, Costa Rica Region
Suggested Condemnation "Risk" Levels

Class	Economic life Yrs.	RP_C	P_C	"Risk"/yr.	A_C g units
1	100	1000	.1	.001	0.35
2	50	500	.1	.002	0.31
3	20	100	.2	.01	0.18

For both the Managua and San José regions, Table 5 gives the values of d_T, d_{OT} and $(1+k_T V_S)$ as functions of the structural types. Table 6 gives the value of H (see Figure 13) for class 2 structures, Managua region. Similarly, Table 7 gives the value of H for class 2 structures in San José, Costa Rica, region. It should be mentioned that using the methodology presented in this report and for the seismicity of Managua, the seismic design level comes out to be somewhat similar to the 1976 UBC. Similarly, for the San José region and its seismicity, the recommended seismic design levels are somewhat similar to the 1973 UBC.

In Tables 6 and 7, $\bar{\mu}_C$ and $\bar{\mu}_{COT}$ are defined as follows:

$$\bar{\mu}_C = \frac{CDS}{DFS} = \frac{A_C}{A_D} \cdot d_T \tag{5}$$

$$\mu_{COT} = \frac{CDS}{DMS} = \frac{A_C}{A_D} d_{OT} \tag{6}$$

It should also be pointed out that the load combination and load factoring are done in the proposed method in the following way.

Table 5

Factors for Design Spectra

Type	β_T	Plateau Value of MDAF	d_T	d_{OT}	$(1 + k_T V_S)$
0.67A	10%	2.0	3.0	3.0	1.0
0.67B	10%	2.0	3.0	3.0	1.2
0.67C	10%	2.0	3.0	3.0	1.4
0.80A	10%	2.0	2.5	3.0	1.2
0.80B	10%	2.0	2.5	3.0	1.4
0.80C	10%	2.0	2.5	3.0	1.6
1.00A	10%	2.0	2.0	3.0	1.2
1.00B	10%	2.0	2.0	3.0	1.4
1.00C	10%	2.0	2.0	2.0	1.6
1.33A	10%	2.0	1.5	3.0	1.2
1.33B	10%	2.0	1.5	3.0	1.4
1.33C	10%	2.0	1.5	1.5	1.6

Values suggested here are preliminary.

In Tables 6 and 7, $\overline{\mu}_C$ and $\overline{\mu}_{COT}$ are defined as follows:

$$\overline{\mu}_C = \frac{CDS}{DFS} = \frac{A_C}{A_D} \cdot d_T \qquad 5$$

$$\overline{\mu}_{COT} = \frac{CDS}{DMS} = \frac{A_C}{A_D} d_{OT} \qquad 6$$

It should also be pointed out that the load combination and load factoring is done in the proposed method in the following way.

Table 6

Factors for Design Spectra

Managua - Class 2 Structures

Type	H	H_{OT}	$\bar{\mu}_C$	$\bar{\mu}_{COT}$
0.67A	0.163	0.163	3.86	3.86
0.67B	0.196	0.196	3.86	3.86
0.67C	0.229	0.229	3.86	3.86
0.80A	0.236	0.165	3.22	3.86
0.80B	0.275	0.197	3.22	3.86
0.80C	0.317	0.229	3.22	3.86
1.00A	0.294	0.197	2.57	3.86
1.00B	0.343	0.229	2.57	3.86
1.00C	0.392	0.262	2.57	2.57
1.33A	0.391	0.195	1.93	3.86
1.33B	0.456	0.229	1.93	3.86
1.33C	0.520	0.520	1.93	1.93

$$H = (0.7)A_D\frac{(MDAF)}{d_T}(1 + k_T V_S)$$

$$H_{OT} = (0.7)A_D\frac{(MDAF)}{d_{OT}}(1 + k_T V_S)$$

Spectrum = H for $T \leq 0.5$ sec
= $.5H/_T$ for $T > 0.5$ sec

For Hard to Medium soil conditions

= H for $T \leq 0.8$ sec
= $0.8\frac{H}{T}$ for $T > 0.8$ sec

For soft sites

Table 7

Factors for Design Spectra

San José - Class 2 Structures

Type	H	H_{OT}	$\bar{\mu}_C$	$\bar{\mu}_{COT}$
0.67A	0.084	0.084	5.16	5.16
0.67B	0.101	0.101	5.16	5.16
0.67C	0.118	0.118	5.16	5.16
0.80A	0.121	0.085	4.30	5.16
0.80B	0.141	0.101	4.30	5.16
0.80C	0.163	0.118	4.30	5.16
1.00A	0.151	0.101	3.44	5.16
1.00B	0.176	0.118	3.44	5.16
1.00C	0.202	0.135	3.44	3.44
1.33A	0.202	0.100	2.58	5.16
1.33B	0.235	0.118	2.58	2.58
1.33C	0.267	0.267	2.58	2.58

$$H = (0.7)A_D\frac{(MDAF)}{d_T}(1 + k_T V_S)$$

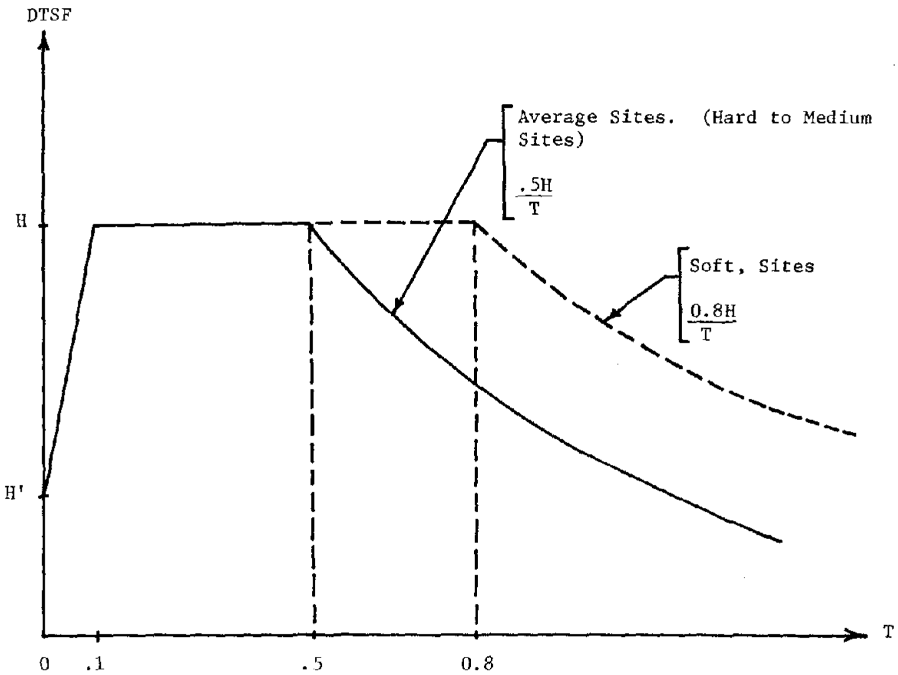
$$H_{OT} = (0.7)A_D\frac{(MDAF)}{d_{OT}}(1 + k_T V_S)$$

Spectrum $= H$ for $T \leq 0.5$ sec
$= .5H/_T$ for $T > 0.5$ sec

For Hard to Medium soil conditions

$= H$ for $T \leq 0.8$ sec
$= 0.\frac{8H}{T}$ for $T > 0.8$ sec

For soft sites



$$H' = \frac{R\ A_D}{d_T}(1 + k_T V_S)$$

$$H = \frac{R\ A_D}{d_T}(1 + k_T V_S)\ \mathrm{MDAF}$$

For design overturning moment, replace H with H_{OT} and d_T with d_{OT}.

For very soft sites, special site study needed.

Figure 13

III.2 Seismic Weights, Load Combinations, and Load Factors

One basic principle that has guided the formulation of the proposed design procedure is that each step and parameter be rational. Specifically, there must be a simple, rational explanation and reason for each representation of seismic input and the corresponding structural behavior. The subject of load combinations and load factors provides a good example of this direct representation approach. Current code provisions will be stated for comparison.

- Seismic Structure Weight or Mass. At the time of the earthquake events corresponding to the PGA values of A_D or A_C, a realistic yet reasonably conservative value must be assigned for the total structure weight or mass, for the evaluation of inertia forces. Some amount of live load is to be expected, and the judgment value of 40 percent is suggested.

  Therefore, for dynamic analyses and for simplified base shear methods the weight or mass is dead load plus 40 percent live load $(D + 0.4L)$. Present codes employ dead load only, except for warehouse structures.

- Load Combinations and Load Factors. Since the selected value of 40 percent live load is quite conservative for most structures in the sense that it is highly improbable that vertical live loads would exceed this value at the time of the earthquake, the load combination for the ultimate strength design R_u of members is dead load (D), 40 percent live load (.4L), and seismic forces E, due to the SRSS response to the Design Force Spectrum (DFS).

$$R_u = D + 0.4L + E \tag{7}$$

In Equation (7),

R_u = the required ultimate strength capacity for this specific case of loading. (Other cases may be for vertical load only, such as $(1.4D + 1.7L)$.)

D = the member force (such as moment or shear) due to dead load.

L = the member force (such as moment or shear) due to the code-specified value of live load.

E = the SRSS of the individual mode member force (such as moment or shear) due to the DFS.

While it appears at first glance that there are no load factors used in this ultimate strength load combination, these do exist. The purpose of load factors is to account for the chance of high possible loads and for differences between analysis and actual structure response. In the load combination of Equation (7), the 0.4L is a reliable upper bound for vertical load uncertainties, and the value of E contains its DFS. It should be noted that each factor is applied directly to the source of load uncertainty. This can best be appreciated by a comparison with current code load combinations, such as

$$(R_u)_{code} = 1.4(D + L + E_{code}) \tag{8}$$

where E_{code} is due to $V = KCS$. In this combination of Equation (8), the safety or reliability of the number design for seismic resistance can vary according to the proportion of vertical to seismic load. For large $D + L$, the section may be overdesigned, and for small $D + L$ the section may be underdesigned, since $1.4E_{code}$ is only about one-half of reasonable damage level earthquake loads as represented by the DFS.

In order to account for the effects of vertical ground acceleration on the lateral force requirements, the following combination is used:

$$R_u = 0.8(D + E) \tag{9}$$

Here, the most critical load condition, for overturning moment tension effects, occurs when there is only a small amount of live load. The 0.8D represents both the reduced dead and live load (due to vertical acceleration). The 0.8E reduces E corresponding to the small live load contained in the structure seismic mass, and also represents the smaller horizontal acceleration at the time of maximum vertical acceleration.

Preliminary computations have indicated that, in moment-resisting frames (and perhaps braced frames), the load combination of Equation (7) may in some cases lead to axial column loads which are significantly smaller than those of the 1973 UBC. This problem needs to be pointed out and requires further study. To account for possible effects of vertical accelerations, it may be advisable to apply a load factor to $D + 0.4L$ for such vertical elements.

III.3 <u>Design Procedure Rules</u>

In this section a step-by-step procedure for the complete design sequence is given.

1. Given a use class of the structure and its location, the values of A_D and A_C can be determined from an iso-acceleration map or the acceleration zone graph. The appropriate design spectra can be constructed with the above information together with the parameters MDAF, V_S, d_T, d_{OT}, and k_T of a given structural type and soil condition.

2. Formulate the linear elastic structure model and determine mode shapes and periods. Then, using the DFS developed in (1) above, obtain the SRSS force response E in the structural members.

3. Design members for load combinations on an ultimate strength basis for the following conditions:

   a) Load-factored vertical dead and live load: $1.7(D + L)$.

   b) DFS or DMS force plus vertical dead and live load: $(D + .4L) + E$.

   c) $0.8(D + E)$ for vertical acceleration effects.

   In (b) and (c) above, the seismic load E is based on a $(D + 0.4L)$ seismic weight of the structure.

4. Interstory drifts using the DDS are calculated as the SRSS of the individual modal drifts. These drifts shall not exceed 1% of the story height. This drift limitation is for damage control.

5. The member design procedure has produced known values for the individual member resistance values, R_u, where

$$R_u > (D+0.4L) + E \; ; \quad R_u > 0.8(D+E) \; ; \quad R_u > 1.7(D+L) \tag{10}$$

and commonly exceeds these load combinations because of the available section or sizing requirements, as shown on the engineering planrs for construction.

For further details, see Shah et al. (1976).

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AN OVERVIEW OF USER NEEDS FOR IMPROVING EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION

by

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INTRODUCTION

This paper is intended to introduce the subject of user needs, and to direct attention toward improvements in earthquake-resistant reinforced concrete building construction as they may be developed to serve users in less seismic areas of the eastern United States.

Every mountain climber harbors dreams of Mt. Everest or K2, and every structural engineer in the building field has visions of designs to rival the Sears Tower in Chicago or the World Trade Center in New York. Similarly, every professional in the field of seismic engineering harbors thoughts of a Richter magnitude 8.5 earthquake having an epicenter under the heart of his favorite city. Professionals who travel far and wide at the quiver of a fault, invariably return with tales of disaster resulting from overlooked conditions of structure response. With this type of concern over catastrophe and destruction, little attention has been devoted to the more mundane aspects of user needs, either in terms of design procedures or in terms of the fundamental philosophy, as they might be effectively met for users of the product in localities where catastrophe is less imminent. The result of this has been an overkill. Users in the less seismic areas sense no real need for such grave concern. They do recognize the possibility of having a complex technical problem introduced into their area of responsibility together with a resultant increased design and product cost, the benefit of which is not apparent.

U.S. Geological Survey open file report 76-416 1 , contains the latest Algermissen and Perkins map of horizontal acceleration in rock (Fig. 1), which shows the regional differentials across the United States, indicating generally that, except for an area in the vicinity of Memphis, no impressive seismic conditions exist in the eastern two-thirds of the country. Proceeding from this information, users as a group in the eastern part of the United States have little reason to feel concern with regard to seismic protection.

At the fall 1976 meeting of ASCE in Philadelphia, Dr. T. T. Fujita 2 presented a map (Fig. 2) showing the occurrence and path of tornadoes in the United States over the past 44 years. It is obvious that communities in the western United States have essentially no interest in this subject, and no incentive to take special precautions to mitigate hazards in building construction which might be attributed to tornadoes. Certainly the Uniform Building Code does not address the issue.

An unwritten but frequently expressed basis for use of the SEAOC "Recommended Lateral Force Requirements and Commentary" 3 , commonly known as the Blue Book, is that the seismic design recommendations are intended to apply to

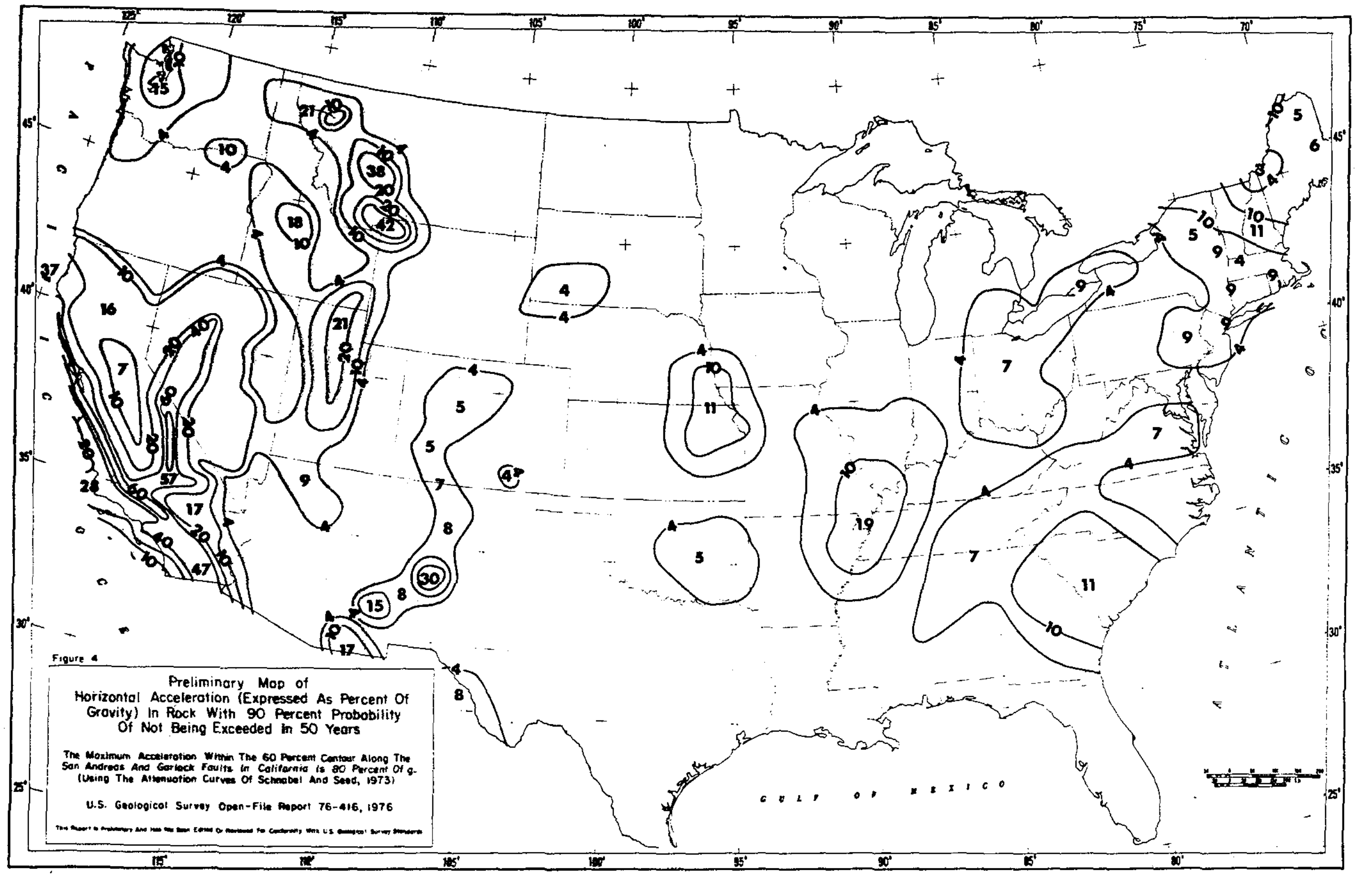

Fig. 1. Map of relative acceleration potentials across the United States.

ordinary buildings, but they are not intended to apply to the extraordinary, such as very tall or irregular structures. Very tall is not defined by story or aspect ratio, but appears to indicate that the designer should consider extra precautions in design above about 5 to 1 aspect ratio or 125 to 150 feet (38 to 46 m) height limits. The document does well in protecting buildings within this envelope. At the same time, the designer of the small structure, warehouse, office building of one to three or four stories, which have irregular configurations may find himself with limited guidance, at the mercy of local authorities in interpretation, or even pursuing an unsafe course.

USERS AND THEIR NEEDS

To make the title significant, users must be identified, together with the relationship of their needs. Engineers ordinarily consider themselves the primary users, faced with the responsibility of designing around the capabilities of the material. The actual list of users who are affected by the material and who are instrumental in its selection for use or in the control of its use is much broader. Architects frequently play a strong part in selection of the material because of its flexibility in forming and potential for architectural treatment; they also establish member shapes, configuration and location in the interest of aesthetics. The building contractor is faced with physical installation of reinforcement and concrete to achieve both the functional ends required by the engineer and the design features desired by the architect; he also has the economic aim of maximizing profit through simplifying construction procedures. Special inspection agencies, particularly the inspectors themselves, must understand the necessary controls and the tolerances allowed to assure that the contractor performs to the degree required by the needs of the engineer and the desires of the architect. Building departments require simple and straightforward regulations covering design limits and materials, that can be administered without ambiguity and, at the same time, without unnecessary constriants on the builder or the designer. Fire protection groups place restraints on material characteristics by establishing protective cover for reinforcement, limitations on aggregate properties and constraints on material thickness or minimum sizes to satisfy fire protective needs. Model code bodies require supportive technical information to assist in development of regulatory provisions in their documents, as do many federal agencies who prepare independent design guides relating to construction within their jurisdiction. The supplier of the materials, while not in the same sense a user, has equal need for knowledge with regard to the capabilities and limitations of his materials or components and the methods by which they can be used most effectively. Beyond this, we have the individual or corporate owner of the completed product with a need for a construction system resulting in an economically attractive investment which might be able to experience moderate earthquakes with reasonable repair potential. Finally, the using community needs a finished product which will continue to serve and add to the community value, while functioning in a manner that does not endanger the citizens of the community.

The owner in many cases is an unknown or variable quantity, and it is impracticle to aim an information program at him. The community is a political body, primarily having a real need for guidance in the adoption of a well prepared code which will provide the necessary freedoms and protection desired. Fire protection groups establish a part of the parameters within which design

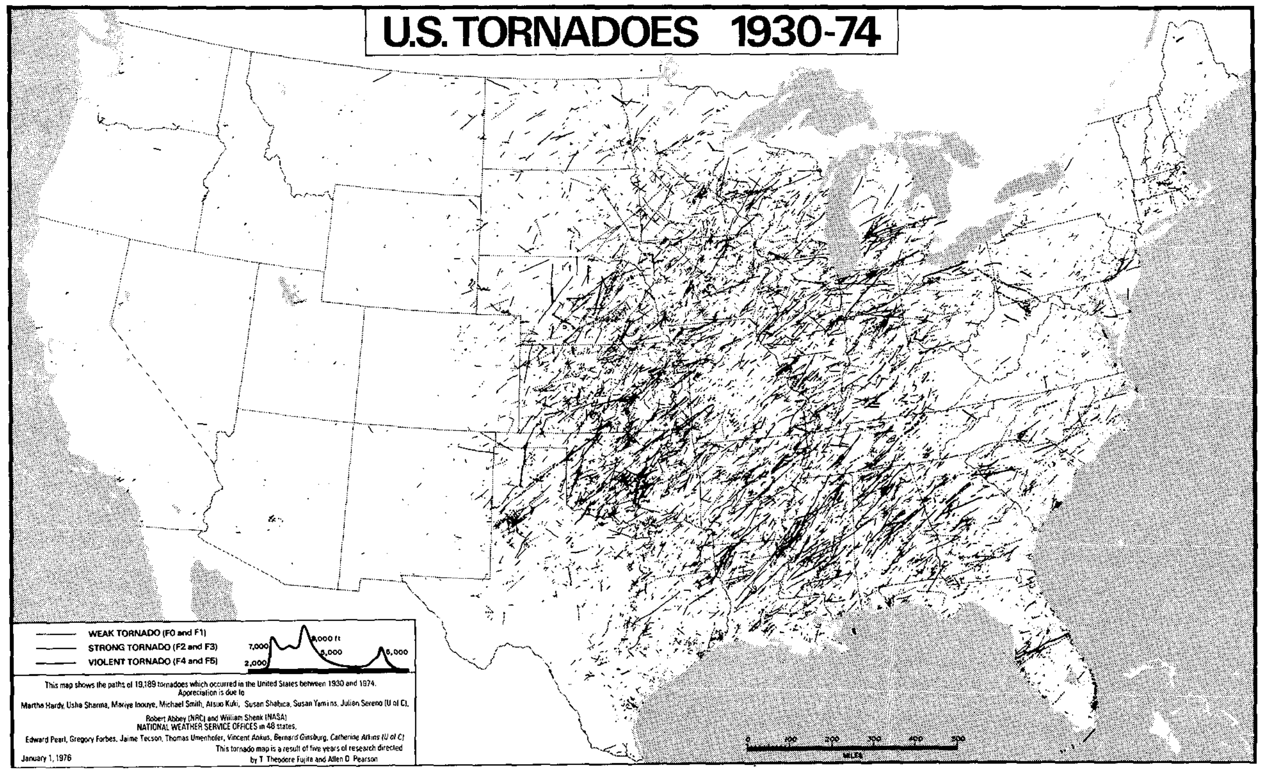

Fig. 2. Paths of the 19,189 tornadoes confirmed in the United States during the 45-year period, 1930-74.

is performed, and these may be independent of seismic considerations. All of the remaining users need to know or appreciate the general character of the research which has been performed and the significance and application of the results to practical problems. Insofar as possible, this communication should flow in a lucid manner from the experimental and analytical field to the interested user, with emphasis varied according to the manner in which the information might serve the user.

THE ENGINEER USER AND REGIONAL CONSIDERATIONS

The Engineer And His Interest

The individual who goes directly from the baccalaureate level to the labor force is still numerically dominant in the profession, and, in some areas, even favored by the hiring office. These engineers with no experience in research and sometimes limited mathematical capabilities, are doing the bulk of the building design, are registered, and practice in a relatively unrestricted manner.

The engineer is not unwilling to perform better work or to use new methods, although he may require some real assistance in making a change. He has a good grasp of problems of practice which may be completely unrelated to research; he is eager, through necessity if for no other reason, to develop effective and economical structures, and his enthusiasm for professional achievement parallels that of the researcher. Being sensitive, he tends to resent the implication that there is something superior about the accomplishments of the individuals in a different field, which sometimes is reflected in a tendency to belittle academic efforts. At the same time, he is the channel through which the results of research are not only put into practice, but also introduced into the area of other user needs, as in model codes.

In highly seismic areas, agreement among engineers is extremely conditional with regard to design methods and regulatory provisions. When an obvious lack of unanimity exists among those faced with a need to be most knowledgeable, the engineers from less seismic localities can scarcely be faulted for objecting to the imposition of severe restrictions.

The Regional Needs

All of this is regional and provincial. The interest of the practicing engineer in any locality is drawn to the problems which he is faced with solving, in accordance with the importance that these matters may be assigned in that region or locality. Research and development on the basis of protection against frequent recurrence of highly damaging earthquakes has not been a matter of apparent concern to the public in the east and the middle west. As a result, design methods developed in more seismic regions appear to place an unnecessary burden on the designer in the less seismic regions against conditions which are either extremely unlikely or completely unprecedented.

Regional interest or disinterest in relating structural practice to seismic risk varies exponentially with the seismic potential of the area, and with the time elapsed since the last seismic occurrence. It is also related to public policy and the philosophy of application of the code, and to the existence of other more frequent hazards damanding the attention of the community. Where seismic design has been undertaken in the United States, it has frequently been based on the principles and philosophy propounded in the SEAOC Seismology Committee Blue Book 3 . This is developed on the principles that a structure should be able to:

1. Resist minor earthquake without damage.

2. Resist moderate earthquake without structural damage, but with some nonstructural damage.

3. Resist major earthquakes, of the intensity and severity of the strongest experienced in California, without collapse, but with some structural, as well as nonstructural, damage.

Designing within the risk maps which have been currently used, the last stricture applies to the severity anticipated in the locality of the structure. While the provisions and principles are sound, they have been propounded for an area where earthquakes have been both frequent and damaging, and where both the public and governmental bodies, as well as the construction community, has been made aware through physical reminders of the potential. When we consider the hazard potential in Los Angeles or the Bay area, and realize how slowly steps proceed to reduce that hazard, which is so obvious and real, there is little wonder that in middle and eastern America engineers show a lack of enthusiasm.

The east does have limited areas with prior seismic experience and significant damage potential, such as Memphis, Charlestown, and Boston. It also has geological characteristics which affect attenuation in a different manner from that experienced on the west coast. Possibly the three-point philosophy has some continuing merit in the locations where greater risk is implied or higher accelerations anticipated, but should be reviewed in lower risk areas. In such areas, static analysis procedures commonly in use for buildings of nominal proportions result in the determination that wind pressures, also applied as static loads, would exceed the loads developed through the seismic force route. Wind loads, recognizing vortex shedding, buffeting from adjacent structures and torsional effects, are nevertheless normally approached as a unidirectional loading conditions, not subject to the rapid reversal of forces experienced in seismic activity at whatever acceleration level. The designer must be convinced that a rapid reversal of forces through a series of cycles has a greater damage potential to the structure than the less frequently repeated and generally unidirectional effect peak wind loads. In heavily seismic regions, the engineer is aware that he is designing for an effective acceleration rather than the actual peak acceleration that might be obtained through instrumental measurements, and that ductility and excursions into inelastic areas provide him with the energy dissipation capability to do this. Acceleration maps currently proposed for application to the entire United States are based on the theory of comparable risk with the application of an acceleration less than the instrumental peak. Thus the designer in the less seismic areas selecting an effective acceleration from the map and developing the resultant equivalent static forces of earthquake may find that wind loads appear to cause greater stresses, without realizing that these effective accelerations may be exceeded substantially, and require structural capabilities beyond those needed to resist wind forces.

This does not mean that there is a great need for extensive effort in the area of seismic design to satisfy the requirements of middle America. It could even be that some alteration of Chapter 7 of ACI 318-71 [4] would suffice to improve the resistance capability of all structures in such a manner that the problem of seismic design in many areas might be greatly minimized or even eliminated.

Uniform Building Code [5] requirements assign importance factors to certain buildings, thus increasing the forcing function. The Applied Technology Council in a pending document assigns buildings to categories and makes detailing more restrictive for buildings with higher resistivity demands, without changing the forcing function. The system of categories appears to be an improvement over percentage increases in forcing function; since the input ground motions will not change.

Category assignment might be reviewed, coupled with a review of the three-point SEAOC Blue Book philosophy, which may be valid in highly seismic areas but is possibly not applicable across the entire country. Should areas of low seismicity and low recurrence have a concern with anything except collapse prevention? In these areas, are we concerned with drift and nonstructural damage except as it impairs egress, or is elementary life protection the only criteria? The Uniform Building Code admits to property concern without expressing any limit. Through experience, structural designers in the west are aware of property damage and invoke drift limits and other restrictions as preventive measures. Code authorities in other areas feel no responsibility for any protection beyond that of life safety.

How do these questions relate to research relative to structures subjected to accelerations of 10 percent to 15 percent g as peak acceleration in 50 years as opposed to 40 percent to 50 percent g in more seismic areas?

In preventing collapse, how do we view recurrence? ATC and current maps of peak acceleration and velocity are based on a 50-year life expectancy for recurrence, although it is unlikely that many reinforced concrete buildings constructed today that are five stories or more in height, will have outlived their usefulness within that period. Much older buildings, in active commercial use that are evident and significant earthquake hazards, are presently fully occupied and some even preserved as historical landmarks.

Should the philosophy of design be directed toward life expectancy of 100 or 150 years with design for noncollapse from seismic events of much longer recurrence interval? How would this relate to design levels considered acceptable in highly seismic areas?

In all areas, there is a tendency to use higher strength material, and, in the middle west, this apparently has resulted in the use of cast in place concrete having strengths in the 10,000 psi (702.9 Kg/cm^2) range. Most experimental work done in the past has been on the basis of a structural concrete practice using material having compressive strengths of 4,000 psi (281.18 Kg/cm^2) and less, combined with grade 40 and grade 60 reinforcement. Utilization of higher strength material may in turn change the ductility and effect the type and nature of failure. Because higher strength material may reduce the member size required, one of the users, the architect, will aim at this advantage while the builder will be pleased at any opportunity to economize on materials.

When the physical experience of an earthquake is close at hand, it is very impressive. The results of such experience are inherent in the professional registration procedure in California, stressing seismic design. In this area, immediate experience is recent and its recurrence is expected; Santa Rosa in 1969, San Fernando in 1971, Eureka and Oroville, both in 1975, so that all users are conscious of the problem. In the east, experience close at hand is lacking, and what happens in California is of the same level of interest as news of China or Italy. Even in the Pacific Northwest, with damaging earthquakes involving loss of life in 1949 and 1965, users and the public remain unimpressed.

First the community must be convinced a concern exists and should be dealt with. This may not be in the area of laboratory research, but it is an essential process if a problem does exist and is to be acted on. This is true, regardless of the level of action required.

RECENT RELEVANT RESEARCH

To say that none of the recent research is relevant to users in less seismic areas may not be true. There is sufficient truth, however, that some effort would be justified to cause researchers to speak more clearly to the needs of these users.

This paper does not attempt to incorporate a detailed bibliography of reinforced concrete research in the seismic area. An excellent source of information for such material is the Report No. EERC 75-12 of May 1975, entitled "Earthquake Engineering Research Center Library Printed Catalog" 6 The volume of published material is large, with much related to reinforced concrete. A later summary listing entitled, "Research That Has Been or Are Being Conducted and Are Related to 'ERCBC'" 7 , prepared in April 1977 by Professor Bertero, includes references to research which is more recent than that contained in the library catalog.

The National Science Foundation in 1976 awarded earthquake engineering research grants exceeding $6,500,000, with a large share assigned to reinforced concrete structures. Within some of these programs will be material of interest to designers in lower seismicity localities, particularly where different levels of ground motion input are used to determine different detailed requirements. An important aspect of this research is dissemination of the information and incorporation of the results in design practice and construction. It is interesting that one of the objectives of the earthquake engineering subelement of NSF is to "present program results in forms usable by the affected interest communities to control the vulnerability to earthquakes".

Scanning research publications soon discloses that this material is aimed at conditions outside the field of interest of the practicing engineer responsible for the bulk of the design work and construction in areas of lower seismicity, if we consider that "bulk" refers to the many low and intermediate rise structures built for general noncritical use. This is not to infer that buildings of critical occupancy, unusual configuration or greater magnitude do not benefit from the level of the research publications available. In these cases, however, the designer is more likely to be aware of a need or may be required by the regulatory body to apply more rigorous design methods.

Picking through published research findings, there are many statements, not necessarily a part of the final conclusions, that can serve as informal guidance to the designer. Statements to the effect that truss analogy for shear function loses its significance under cyclic loading and is not significant in earthquake resistive design. Statements regarding the significance of steps to avoid brittle failure and to attempt to guide failure mechanisms into areas where repair is feasible are matters which are incidental to findings in severe earthquake conditions. These may be helpful to the development of satisfactory details by the designer under less rigorous requirements. Consideration of shear wall action and frame action (separate or combined) is investigated against artificial spectra or historical spectra, such as El Centro, creating an extremely rigorous condition. Little has been contained in these research efforts with regard to variations in earthquake intensities, although the designer in low intensity areas needs such guidance.

THE IMPACT OF RESEARCH

Introduction To Building Codes

In terms of the model code groups, neither the Southern Building Code 8 nor BOCA Standard Building Code 9 provide evidence of any impact of research. On the contrary, both are either noncommital or badly outmoded with respect to seismic design. Both bodies have, however, participated in the current ATC-3 project for development of seismic design provisions. The American National Standards Institute in its standard A58.1 reproduces a slightly outdated image of the Uniform Building Code.

The Uniform Building Code, which serves the western and seismic parts of the country, contains the most advanced seismic code information that has been developed in the United States. This is not a direct result of research activity, but rather largely the result of the impact of the Structural Engineers Association of California through its Seismology Committee. As a result, it represents an input of information and guidance that has been found useful and workable by practicing designers. Nevertheless, on the west coast, designers and research engineers are quite closely allied and what really appears is that the Uniform Building Code reflects the result of research filtered through the experience of practice.

An area which does reflect more direct input from research is that of ACI 318-71, which has incorporated Appendix A as the result of efforts on the part of committees whose members represent both research effort and practice. This appendix is limited to ductile frames and shear walls having boundary elements, and fails to respond to seismic design needs for conditions of lower seismicity or where the special requirements of ductile frames are not required.

Impact of research on codes is only achieved when the results of research have been clearly and successfully disseminated to the users and accepted. Modification for adoption of reserach recommendations can be approached through the practicing engineer who can provide assistance in securing code changes where they are required. It requires the interested cooperation of concerned engineers, faced with real problems, since few of the other parties to the construction process have the same interest in modifying the rules under which the regulating body functions. This is particularly true if there is added effort or expense to be considered.

The Uniform Building Code bears the impression of practicing engineers because the Structural Engineers Associations of the western states have concerned themselves with code activities. These are state groups only, and not a national body, and thus are provincial in their interests. The American Society of Civil Engineers, which is the national body representing structural engineers, has until recently avoided all formal connection with code activity. This policy has changes, but it is unlikely to result in a rapid increase in code interest, until the normal inertia of a large organization is overcome.

The Path of Research Use

Research information finds its way into published reports which may appear in the ACI Journal or the ASCE Structural Journal, or in documents published in conjunction with sponsored research grants. These publications, of course, are not an end, but serve as an intermediate step in putting the findings to use by the profession. One of the most effective examples of transferring research to method and thence practice is that of the 1961 publication by the Portland Cement Association of the Newmark-Blume-Corning book "Design of Multistory Reinforced Concrete Buildings for Earthquake Motions" 10 . The extensive knowledge and experience of the authors together with extensive reference to published material dealing with seismic problems and experimental research, established an acceptable design procedure and guide within a single publication. The broad and gratuitous distribution of the book, with explanatory seminars by the Portland Cement Association put the results of research and experience directly into the hands of interested users. It was not, however, the end-all to research or design needs, but rather a state-of-the-art document prepared at a time when general structural design procedures were being modified; the characteristics of the structures themselves were being architecturally altered; and material characteristics and capabilities were changing.

Research has been performed covering investigation of reinforced beam-column connections for earthquake, behavior of flexural members and behavior of frames and framed structures, flat slab structures and structural walls.

Investigations have been made of structural failure in earthquakes, both in the United States and elsewhere, by Earthquake Engineering Research Institute teams, and relations between materials, construction procedures, design methods and the resulting earthquake problems have been reviewed. A limited audience has been reached with some of the findings, but this consists primarily of users concerned with highly seismic conditions.

The list of institutions performing research is not limited to the highly seismic areas of the country, although an understandably high level of interest leads to the development of much seismic research in California. At the same time, earthquake engineering reserach is being pursued in fifteen universities throughout the United States by the Portland Cement Association at its laboratory in Illinois and by the Research Division of John A. Blume and Associates. Academic interest in the seismic problem in less seismic portions of the United States does not necessarily transmit an interest in seismic safety problems to the public, building authorities, architects, or engineers in these areas.

ACI 318-71 provides the reinforced concrete design guidance used by many engineers. It responds to research in its intermittent publication, and provides a careful, deliberate path to changes embodying research results. Numerous committees have representation from those performing research. And what is the result? An outcry concerning the lack of workability, academic orientation, and overburdening of the designer. Perhaps the complaints are overblown, and perhaps they have died down with familiarity. Certainly none of us enjoy being moved out of a comfortable rut, and this generation of engineers has seen several changes in design technique. Underlying this, however, is the possibility that a large percentage of individuals in the engineering field are unable to understand the material, or are unwilling to make the effort. On the other hand, perhaps they are not being provided with sufficient guidance to help them assimilate new and useful material because of the gap between research effort and design.

In a more general area, much of the result of research is published in the ASCE Engineering Mechanics or Structural Journals. These journals are subscribed to by over 20,000 members, and yet there are many more engineers performing structural design who either are not familiar with the publications or do not read them. Unfortunately, most of the papers present raw research results, with highly complicated mathematical derivations, and the readership tends to be made up of the limited group publishing. There is a place for these publications as long as they are being scrutinized, digested, and the results incorporated into practice where warranted. But the gap that exists between this material and the practicing designer is broad, resulting in polarization rather than communication. The gap must be reduced to assure that worthwhile advances in the state-of-the-art are incorporated into the state-of-practice.

Beyond these generally available areas of material, there is a vast volume of research published which appears to be completed and buried, unless it is searched out, or one is on the proper mailing list. Certainly not every engineer has either an interest or a need for this material, and much of it can be easily traced through catalogs and bibliographies such as those at the library of the University of California at Berkeley and Earthquake Engineering abstract journals. While the practicing designer does not need contact with this material, it is desirable that the useable and practical portions of the material should be adapted to design criteria measures and introduced into the stream of design.

RESEARCH NEEDS

A report was published in September 1975 [11] identifying research needs and directing the attention of the research fraternity to areas of some general applicability, but also largely toward structural problems much more severe than those envisioned by the user in the less seismic areas. These latter users have had little direct impact on research, however, it must be emphasized that they represent a geographic area much larger than the highly seismic area of the country, and a total construction volume and population greatly exceeding the construction volume and population occupying geographic areas of high seismic risk. This creates a valid reason for investigating the steps that are necessary, economically acceptable and easy of accomplishment, to assure that all of this construction in the largest part of the country is prudently accomplished, and that safe conditions are reasonably assured.

In the pending Applied Technology Council ATC-3 document, 12 categories have been established as functions of the nature of concentration of the occupancy in terms of exposure hazards, and of the local seismicity in terms of an index number relating it to hazards in other geographic areas. Under the least demanding category come those buildings, the treatment of which is essentially limited to wall anchorage and tie requirements, to improve seismic resistance. In the next lowest category, the requirement, among others, is that "concrete frames must be semiductile with some transverse reinforcement in the joints". Users have a very great need for an interpretation of this rather open requirement in localities such as Bend, Oregon; Bismark, North Dakota; or Birmingham, Alabama; where seismicity and the hazard index are low. Even areas of greater hazard, large buildings in this category, calling for semiductile frames, will be accepted for moderate occupancy and noncritical conditions. This intermediate level of treatment, and the effectiveness of reinforcing and detailing has not been investigated, or, if it has, the resulting definition of details and reinforcement material characteristics, together with the ductility relationship, have not been disseminated in terms of meaning to the practicing engineer.

There is an accumulation of hysteresis data based on severe cyclic load experience, establishing a definition of ductility, and its needs under extreme conditions. Since less seismic regional design permits lower effective peak accelerations and velocities, with shorter durations of shaking, research should investigate the effectiveness of modified levels of augmented reinforcement under related input to secure a pattern of effectiveness which can define semiductility and asses member reinforcement needs.

Regardless of geographic locality, the problems of definition of irregular structures and simplification of analysis of their response has not been solved. The knowledgable designer will show greater concern as his structure becomes less regular in its conformation and may enlist the sympathy of the architect to reduce these irregularities in some measure. At some undefined point, dependent primarily on the designer's judgment and his experience, he will increase the sophistication of the design measures and treatment of details, taking into consideration the extent of the irregularities and the importance of the construction. Familiarity is necessary to recognize the hazards of irregularity, and in the low seismicity areas of the country, designers involved largely with noncritical construction may not readily appreciate the critical nature of extensive mass eccentricities, setbacks and vertical discontinuities. A rationale for increasing in a simple manner the sophistication of design has not been satisfactorily developed. The definition of irregular is surrounded by such modifiers as "significant", for which we have neither the background nor the stomach to apply numerical assistance. The only limiting factor for change has been the long standing 160-foot limitation on height without ductile frame, which is admittedly arbitrary, but generally applied without latitude, and as an absolute.

When we consider that the present Southern Standard Building Code contains no guidance or restrictions with respect to seismic design, and that the BOCA Basic Building Code still retains the forcing factors which were introduced 25 years ago, together with a seismic zoning map of equal antiquity, it must be apparent that only the most elementary modifications to seismic design practice can be anticipated in the eastern United States, except where it can be clearly and undeniably shown that a hazard will be developed through lack of considering conditions of structural arrangement or shape.

No philosophical or technical assessment of the limits of acceptable damage has been investigated. The problem of repairability of damaged structural elements under recurring conditions is of interest in areas of high seismicity, while the question of noncollapse under the infrequent damaging earthquake is of interest in areas of low seismicity. This also includes the need for information on the strength decay of columns under earthquake loads, where the bearing area of the concrete may be reduced to a mass of broken material basketed in the reinforcement.

Extensive experimental and analytical research is in process in the area of structural walls. It is important to the designer, at all levels of seismicity, that this material should be first carefully examined by the research community, but then carried into simple and useable design terms and actively disseminated to the using engineers and the rest of the building field. As a part of this effort, it is important that a system of design limitations be achieved which will permit rational and realistic assessment of capabilities without the need for arbitrary imposition of height limits.

Further effort appears to be required to determine a highly simplified method for the application of modal analysis. Simple means of calculating modal response and simple guidelines to permit its application and understanding by the average designer without need for incomprehensible analytical devices may be an insurmountable request, but would serve the designers greatly. To this should be added suitable guideposts to establish where the method is significantly useful or even necessary.

Studies of floor diaphragms, to determine their real stiffness and limitations applicable to their use, are needed. This should be extended into the area of effectiveness of systems utilized as a floor diaphragm consisting of precast elements either with or without topping. Little information is available with regard to the effectiveness of topping slabs and their transmission of shearing forces into the vertical load carrying elements, or of welds interconnecting these precast units.

These areas are general in nature, but for the most part would provide assistance to designers in low seismicity parts of the country. They do not propose to cover many other questions of possible user needs which are more pressing in west coast states.

CONCLUSIONS

Efforts toward solution of the seismic design problem will and should continue to focus on severe earthquake forces in areas in high recurrence level. This is of primary interest only to a relatively limited geographical area of the country. There is every reason to devote substantial effort to establishing hazard mitigation policy and the necessary related design guidance and construction guidance in less seismic areas. The answers to some of the problems arising under these conditions can be developed from published research relating to severe earthquake effects; however, there are areas of intermediate interest which may require independent investigations. Correspondence with numerous engineers having long connection with seismic design and research has provided suggestions toward user needs, but of more interest, it has provided a surprising independent agreement concerning shortcomings. It is generally agreed that while there is much research performed, the activity is relatively uncoordinated and the results are inadequately transmitted to the potential user. This is reflected in excessive complexity of the existing code as it pertains to gravity loads, as well as the tendency to develop even more complexity in seismic requirements.

There are large and unsatisfactory gaps in communication at all levels of the process. Transmission of research finding and results is too frequently in terms which are incomprehensible to the engineer attempting to profit from them. This impedes adoption of what might otherwise be worthwhile findings, because of unclear and inadequately simplified expressions of those findings. In turn, this material is ineffectively put into practice and its use is impeded or prevented through lack of the further links of communicating with the architect, the builder, and the other users.

In all areas of the country, it is imperative that significant research results be communicated in such a manner to the users that they will serve to improve the effectiveness of the product. In those large areas of the country where less concern exists regarding seismic activity, if some precautions are truly warranted, such communication must be even more simplified and definitive.

In view of the fact that dissatisfaction is expressed with regard to communication between researchers and users, and the assessment by the users that research is fragmented, it would appear desirable to perform a review of past and current work. This should be aimed at segregating the material which can be usefully adopted to practice, and developing an outline of procedure and review through which it might be adopted by users and have an impact on codes. This may be difficult to accomplish without intruding on the private domain of investigation of individual researchers. Hopefully it will not conflict with the self-interest of institutions or individuals to establish some type of overall coordination. At the same time, this should be the result of a free interchange of relavant material without imposition of requirements.

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AN OVERVIEW OF THE STATE-OF-THE-PRACTICE AND OF USER NEEDS FOR IMPROVING ERCBC.

(EMPHASIS ON CALIFORNIA)

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INTRODUCTION

Building design practice is dominated by the codes governing the design since, even though the code intent is to provide minimum standards, in actual practice these minimums generally become almost the only design criteria. Research and testing is also closely linked with codes since the results are generally used only where they influence code criteria. This overview of earthquake resistant reinforced concrete building construction in California therefore revolves around the seismic codes used in California in recent years.

The discussion is divided into five sections.

1. Concrete Shear Walls.
2. Interaction of Shear Walls and Frames in Resisting Seismic Forces.
3. Concrete Floor and Roof Diaphragms.
4. Concrete Moment Frame Seismic Force Resisting Systems.
5. Miscellaneous Topics.

The symbols used in the text are those used in the ACI and Uniform Building Code, unless noted otherwise.

CONCRETE SHEAR WALLS

Since the first SEAOC requirements were included in the 1961 UBC, the seismic lateral force requirements and the shear capacity allowance for concrete shear walls has changed radically. This is most easily recognized if a specific example is used to trace these changes. Consider a building with 100,000 sq. ft. of floor area and a uniform dead weight of 100 psf whose code seismic forces are resisted by 10" thick shear walls with 3,000 psi concrete and .25% of grade 40 reinforcing. The length of shear wall required by the various codes for buildings with a complete vertical load frame and with periods of .5 second and 1 second are as follows:

Code	T = .5	T = 1
1961 & 1964 UBC	35.0'	27.7'
1967 & 1970 UBC	58.3'	46.2'
1973 UBC	83.3'	65.9'
1976 UBC	130.8'	93.4'
Proposed ATC	132.4'	72.8'

This is the result of applying the following force formulas and shear capacity allowances:

For the 1961 to 1973 codes the seismic coefficient formula is $C = .05/T^{1/3}$. This results in a design lateral force of .063W for a building with a period (T) of .5 second, and .05W for a building with a period of 1 second.

The 1976 force coefficient formula is $C = .067\ S/T^{1/2}$. The site factor S is most commonly equal to 1.5, yielding a formula of $C = .10/T^{1/2} \leq .14$, which is used for the above table. This results in a design lateral force of .14W for a building with a period of .5 second and .11W for a building with a period of 1 second.

The current ATC study proposes a seismic coefficient formula of $(1.2 \times .4 \times 1.2)/(5T^{2/3}) \leq (2.5 \times .4)/5$ for concrete shear wall buildings with a complete vertical load frame placed on the usual California site. This results in a design lateral force of .20W for a building with a T of .5, and .11W for a building with a T of 1.

The 1961 to 1964 codes provide an allowable concrete shear value of $.05f'_c$, or 150 psi for the given concrete, regardless of whether the reinforcing exceeds the minimum requirement of .25%.

The 1967 to 1970 codes provide an allowable concrete shear which depends on the wall height-to-length ratio and the percentage of reinforcing. For height-to-width ratios less than 2, and .25% reinforcing, the allowable shear is 90 psi.

The 1973 code provides an allowable shear of $.85(2\sqrt{f'_c} + pf_y)/2.8$, which, for the given concrete and reinforcing gives an effective allowable shear of 64 psi.

The 1976 code provides an allowable shear of $.85(2\sqrt{f'_c} + pf_y)/2$. For the given concrete and reinforcing, the effective shear is 89 psi.

The current ATC proposal provides an allowable shear of $.6(2\sqrt{f'_c} + pf_y)$ yielding an effective allowable shear of 126 psi. Small increases are allowed per the ACI.

If the ultimate concrete wall shear capacity is given by the equation $v_u = 2\sqrt{f'_c} + pf_y$, the various codes would provide for the following lateral force coefficients at ultimate shear capacity:

Code	T = .5	T = 1.0
1961 & 1964 UBC	.088	.070
1967 & 1970 UBC	.147	.117
1973 UBC	.207	.164
1976 UBC	.330	.240
Proposed ATC	.333	.186

A typical design earthquake spectrum would show a 5% damped SDF (single degree of freedom) elastic response on the order of 1.0g at T = .5 and .50g at T = 1.0. For a MDF (multi-degree of freedom system) which represents a typical building, these values would reduce to about .85g and .42g. With the damping increased to 10%, the values would reduce to about .64g and .32g. For most buildings, there are factors, which will be discussed, which prevent the theoretical response force from being delivered to shear walls. However, for buildings which may not have these relief factors, we need to at least reconcile the gap between the 1976 UBC capacities and the design earthquake dynamic response predictions. For buildings with a period of .5 second, the difference is a factor of about 2. For buildings with a period of 1 second, the difference is a factor of about 1.33. These factors have to be reconciled by a combination of shear strength capacity in excess of $2\sqrt{f'_c} + pf_y$, ductility, and damping. For walls with only .25% reinforcing, most tests have indicated very little shear ductility, generally less than 20%. An increase in damping to 20% without wall shear failure is possible, but this is probably accounted for in a ductility allowance of 20%. Static tests, monotonic and reversing, indicate that the ultimate capacity may exceed $2\sqrt{f'_c} + pf_y$ by as much as 50% for short walls, where the concrete is actually taking all the shear. For these walls, however, no shear ductility is evidenced. It seems that the gap between design earthquake response spectra forces and 1976 UBC design capacities can be almost, but not quite, closed on the basis of what we now think we know about concrete shear.

Changes in what we think that we know about concrete shear can be traced through the changes in codes.

Codes previous to the 1976 UBC related shear wall strength directly to concrete compressive strength. A minimum amount of wall reinforcing was required but no credit was given for additional reinforcing. The allowable shear for seismic forces included about the same margin against assumed ultimate as the margin used for other loads. The shear wall allowable of $.05 f'_c$ could be justified on the basis of the $.03 f'_c$ allowed for shear in unreinforced beams, plus an allowance for the wall reinforcing, or could be considered an arbitrary increase for walls over normal beams.

The 1967 UBC changed the allowable concrete shear radically. Concrete shear strength was related to the square root of the concrete strength, rather than directly with the compressive strength. Design was converted to a factored up loading to be used with ultimate capacity, or, considered conversely, an allowable capacity factored down from ultimate. Recognizing the high seismic response forces that shear walls attract if their shear strength governs, the 1.4 seismic "U" factor for concrete was doubled for shear capacity. Greatly complicating design, an attempt was made to reflect the effects of combined shear and flexure. It was assumed that walls with height-to-length (H/D) ratios of 1 or less have an ultimate capacity shear stress of $5.4\sqrt{f'_c}$. For 3,000 psi concrete this assumes an ultimate shear of 252 psi, which, when factored down by 2.8, gives an allowable shear of 90 psi. It was further assumed that shear reinforcing for short walls is completely ineffective, so the maximum allowable shear for these walls was 90 psi, regardless of the reinforcing. For walls with H/D ratios greater than 2.7, the ultimate shear was given as $\phi\ (2\sqrt{f'_c} + pf_y)$. For 3,000 psi concrete and .25% grade 40 reinforcing, this formula gives an ultimate capacity of 178 psi with a factored down allowable capacity of 64 psi. For H/D ratios between 1 and 2, the allowable shear could be increased linearally from 90 psi to as much as 166 psi, depending on the amount of reinforcing. From this maximum, the possible capacities decreased to the $\phi\ (\sqrt{2f'_c} + pf_y)$ value at an H/D of 2.7.

The above H/D shear criteria, which was included in the 1967 and 1970 UBC, are an example of why we should not try to put all that we think we know into design codes. While these code provisions were in effect, designers struggled to compute effective H/D ratios and allowable stresses for an almost infinite variety of piers created by wall openings. The effective height (H) to be used in the formula was almost impossible to define based on the pier end conditions. When the allowable stress complications were added to the involved elastic stiffness calculations which were made to determine the distribution of shears between piers, such involved calculations were made that they actually deterred general accuracy and discouraged checking. The computational complexities were out of all proportion to the possible accuracy of seismic design.

The 1973 UBC changed the computed ultimate shear capacity for concrete walls to $\phi\ (2\sqrt{f'_c} + pf_y)$ for all walls, without changes for H/D ratios. This provides a sum equivalent of about $\phi\ (4\sqrt{f'_c})$ for walls with the minimum required reinforcing. For low (H/D) ratios this could be considered as a decrease in capacity allowance from the previous code to allow for the high response and low ductility of these walls. It was also argued that the value of $5.4\sqrt{f'_c}$ does not apply to the type of load application involved with many shear walls. And it was further justified as a concession to simplifying design, recognizing that the previous ultimate value of $5.4\sqrt{f'_c}$ can be equaled with the $\phi\ (2\sqrt{f'_c} + pf_y)$ formula by simply increasing the percentage of reinforcing. The 1973 UBC maintained the load factor of 2.8, providing a factored down shear capacity of 64 psi for a wall with 3,000 psi concrete and .25% of grade 40 reinforcing.

The 1976 UBC increased the seismic lateral force coefficient so the concrete shear load factor was reduced from 2.8 to 2.0 to provide approximately the same required capacity as the 1973 code.

The proposed ATC uses the formula $.6\ (2\sqrt{f'_c} + pf_y)$ to represent ultimate concrete shear capacity. This provides a capacity reduction factor of 1.67, as compared with the 2/.85 = 2.35 factor of the 1976 UBC, but this is used with the increased lateral shears provided by the ATC seismic force formula.

It can be seen that concensus opinion, as represented by current codes, considers that the essential controllable factors of shear capacity in concrete shear walls are represented in the basic shear capacity formula $v_u = 2\sqrt{f'c} + pf_y$. It is generally recognized that this formula does not necessarily (and does not need to) predict the real distribution of shear capacity between concrete and reinforcing. However, the sum of the resistance indicated by this formula is believed to reasonably estimate the value of that part of ultimate shear capacity which is controllable in the complex design for earthquakes. This is in spite of the many tests which have shown that concrete shear capacity varies widely, depending on a great many variables. The variables include not only the concrete compressive strength and the shear reinforcing, but also the flexure reinforcing, the combination of shear and flexure stresses, and the combination of shear and direct stresses.

Two basic types of static tests have been made to simulate combinations of shear and flexure, "block-loaded" beam type tests, and "flange-loaded" racking type tests. For short deep beams or walls, "block-loaded" tests tend to develop diagonal compression struts which create principal tension cracks perpendicular to the struts. The concrete shear capacity thus indicated is high. "Flange-loaded" tests tend to develop the more usual diagonal tension cracks which generally result from shear forces. Both of these loading conditions can be found in actual buildings, but the most common condition would be that represented by the "flange-loaded" tests. The influence of direct stresses has also been investigated extensively and found to be very significant, if the direct stresses are significant. However, not only are the direct stresses in shear walls generally not very great, but they are also not very predictable. Where the dead load direct stresses are shared with reinforcing, the shrinkage and creep of concrete tends to shift the load to the steel to a somewhat unknown degree. The direct stresses due to earthquake dynamics add to or subtract from the dead load stresses in a complex and far from predictable manner. The oscillating modal vibrations of a building responding to the random unpredictable variations in earthquake ground motions are not represented by the static shear envelope and moment used for basic design seismic forces.

When faced with many relatively unpredictable factors, the cost of neglecting all but the more predictable factors needs to be considered. Essentially, the value of the more unpredictable factors of ultimate shear capacity can be simply replaced by the addition of shear reinforcing. This is not an unreasonable economic penalty and therefore is reasonable justification for the current design shear capacity approach.

Several shear factors have not been discussed in the above.

First, the concrete shear capacity may not only depend on the concrete compressive strength, but also on the shear strength of the aggregate. When aggregate weaker than normal rock aggregate is used, the shear strength of the concrete may be reduced. Reduction factors for lightweight concrete are provided in current codes, based on tests of typical lightweight mixes.

Second is the important factor of horizontal shear along construction joints where, as the ACI code puts it, "it is inappropriate to consider shear as a measure of diagonal tension". Shear failure along these construction joints has been repeatedly demonstrated by earthquake performance. The current ACI "shear-friction" provisions actually call for shear design along these joints to be governed by the formula $v_u = \phi p f_y$ anf for "the interface to be rough with a full amplitude of approximately 1/4 inch". Actually, the provisions were developed from tests tailored for, and intended for vertical load application to corbels supporting precast members, and similar details. There is no indication that the provisions are actually being applied in practice to horizontal construction joints in shear walls and the provisions do not directly relate to the tests. However, there is certainly a need for tests directly related to this problem and for code provisions based on these tests.

Somewhat along the same line as the horizontal crack or joint problem is the vertical crack problem where short spandrels join large shear wall piers. Flexure yielding at ends of the spandrels opens up a vertical crack which becomes a significant separation with reversing loads into the yield range. Diagonal reinforcing cages can transfer this shear and considerable attention has been given to this concept. However, it may be much more economical and adequately effective to transfer this shear through large dowels placed near the center height of the spandrel. This concept needs testing.

Third is the subject of providing shear ductility by using a sufficient amount of shear reinforcing. There is considerable debate about the load factor used for concrete shear. Those who believe that it should be reduced from the present code factor generally assume that shear walls do have significant ductility, though they do not always separate flexure ductility and shear ductility. The subject of flexural ductility will be taken up after this discussion of shear capacity is complete. Regarding shear ductility, however, there is considerable test data available, much of which is from New Zealand. This indicates that considerable shear ductility is provided if the shear reinforcing has sufficient yield strength capacity so that it does not yield significantly with concrete cracking. It is possible that more safety with less load factor could be achieved through a higher minimum shear reinforcing capacity than is now required.

The subject of ductility and period lengthening due to stiffness degradation at high stresses needs examination from the standpoint of degrading shear modulus, for individual piers and for shear wall assemblies. Very little attention has been paid to the shear modulus for concrete on the basis that it is not very significant. At high shear stress for tall walls the shear deflection may be very significant. The few masonry tests where shear modulus degradation with stress and distortion have been recorded indicate that this can be an important factor in the behavior of tall walls. Too often the tests have not concentrated on deflections and the effects of various contributers have not been singled out. Shear walls with openings creating a variety of piers and spandrels have a stiffness and stiffness degradation mixed between shear, flexure, and direct stresses. More of this type of wall needs testing.

It can be seen from this long discussion of shear capacity that many of the code provisions and the wall tests have concentrated on the shear aspect of shear wall capacity. However, the code shear capacity will not be reached, even in buildings subjected to severe ground shaking, if the wall yields in flexure, or rotates on its foundation, or slides on its foundation before the response force builds up to the shear capacity. Some engineers are of the opinion that a shear wall should always be designed to yield in flexure before it reaches its shear capacity, thus assuring the ductile relief provided by yield flexure, as is provided in frames. Others believe that most walls should rotate on their foundations at a low force level to limit shear forces. Certainly, many walls do not have a connection with the ground which will transfer the elastic response forces generated by severe ground shaking. The problem is that many of these forms of force relief are not available for certain buildings and many can not be made predictable. It must be remembered that the structure is serving architectural functions, and many times the architecture dominates, as long as the cost of providing earthquake safety is not made prohibitive by the architectural requirements. And it must be remembered that the architect generally wants the concrete wall elements to be both structural and architectural rather than cladding or in-fill elements.

Tests have indicated, and it has been generally stated, that shear walls yielding in flexure have the needed ductility to reconcile code forces induced by the energy demands of severe earthquakes. There seems to be some inconsistancy here between the design of concrete ductile frames and shear walls. The design force for a shear wall with a complete vertical load frame is only 50% greater than that for a frame (except for shear stresses and diagonal tension stresses). However, the code requirements for assuring flexural ductility for the two systems are not at all proportional to these design forces. The rotation demands on the system are generally very different, but they may be very comparable, for some configurations.

One of the problems in discussing shear walls is the fact that usually each person discussing the subject has a different picture of what shear walls look like in buildings. Those who believe that bending does, or should, always control over shear generally picture a wall pier which cantilevers up from a foundation or basement. For this type of wall, flexure will govern at low height to length ratios. However, the capacity of such walls will usually not satisfy code force requirements without end columns, or end wall returns. For these walls, wall shear yielding may be preferable to column yielding. The shear on these walls at strong ground shaking may be limited by uplift at the foundation, or basement embedment may not provide any limit to the dynamic response. Often shear walls are part of a box configuration, in a core area, or as perimeter walls. Core area walls may get relief from response shears, due to rotation of the box core as a whole, but they will not yield in flexure. Perimeter walls in a box shape will generally get no relief, either from rotation or flexure. Box core walls have openings and the design of piers between such openings brings up new problems. The usual design of jamb reinforcing for the piers may not apply because the rigid box forces diagonal strut action rather than bending plus diagonal tension shear.

The problem of drift and distortion control for walls which rotate on their foundation needs considerable research.

An elastic analysis of shear wall drift is not valid if one end of the shear wall starts to lift off of the foundation. Considering the shear wall by itself, the only drift control at this stage is the balance between kinetic energy input to the wall and the potential energy involved in raising the c. g. of the wall due to rocking. This potential energy control is reasonably effective at limiting drift only for low velocity motions. Since the energy is related to the velocity squared, it takes a large rotation to balance high velocities. The fact that seismic motions change rapidly and are generally reciprocating generally prevents large size elements, such as walls, from overturning even with high velocity ground motions. There isn't time during one motion cycle for the wall c. g. to rotate past the edge about which the wall must rotate. It cannot be assumed, however, that because a wall will not overturn, it will not rock. It may rock on the foundation if the dead load or tie down resistance to rocking is not great enough, or it may rock about a section where the wall tensile capacity is not adequate for the bending moments involved. Of course, a wall will also rock somewhat due to unbalanced soil loading under the footings resulting from lateral loads on the wall. Although this may increase the flexibility and period of the wall, it will generally not increase the period enough to significantly reduce the dynamic response. The rotation settlement involved in doubling the load on one footing while reducing the value to zero on the other is generally quite small.

INTERACTION OF SHEAR WALLS AND FRAMES IN RESISTING SEISMIC FORCES

Shear walls in multi-story buildings often have large height-to-length aspect ratios. These walls will resist lateral forces as cantilever vertical beams whose deflection characteristics depend more on bending moment than on shear. Generally, the building frame, whether designed for lateral forces or not, will act in conjunction with the shear walls. The frame, though resisting lateral forces principally by bending of the frame members, will distort mainly as a shear unit, if, as is usual, its height-to-length aspect ratio is low. Normally the frame stiffness distribution with building height will approximate the lateral load shear distribution with height, resulting in an essentially straight vertical distortion curve. Combining the effect of the bending curve of the shear walls with the straight curve of the frame indicates that almost all of the lateral force shear at the bottom of the building will be taken by the shear walls while the frame will pick up shear at the top.

Computing the distribution of lateral force resistance between the shear walls and the frame is a complicated mathematical analysis problem and a tedious design problem. Computer programs will readily handle the elastic analysis problem but require a lot of man and machine time for the trials necessary in design. Somewhat simplified algebraic methods have been worked up but they are still quite tedious and time consuming. As a result of these complications, the shear walls are generally designed for all the lateral load and the frame either designed for an independent share of the load, or not designed to take any lateral force.

It is necessary, however, in evaluating the performance of buildings which have been subjected to earthquake ground motions, or in evaluating the risks of possible earthquake exposure, to consider the combined resistance of the two systems. For this problem, for realistic seismic design, the post elastic performance of the systems must also be considered. Although post elastic effects can be considered in computer programming, they generally have not been because this is difficult and consumes much computer time. A quicker and less expensive means of evaluating the performance of these combined systems is needed. It seems that a simple semi- graphical mathematical approach will provide a reasonable estimate of the performance of such systems, elastic or inelastic. The principal inelastic effect is wall rocking. Normally, if a shear wall starts to rock it will start to lean on the building frame and the frame will support it up to the capacity of the frame. Expressed graphically, this is indicated by a sharing, between the two systems, of the lateral load diagram and its resultant shear diagram.

If the walls have been designed strictly to code minimum requirements with regard to overturning moment and dead load or tie down moment resistance at the foundation, they will rock if subjected to forces in excess of the code minimum. This, of course, can be recognized as a definite possibility, if not a definity probability. For a shear distribution which approaches the usual parabolic assumption, the rocking moment involved is equal to the base shear times two-thirds of the building height. Since the balancing resistance moment to rocking is a quantity fixed by the design, any increase above the design base shear due to more intense ground motion must be balanced by a decrease in the effective moment lever arm. Since moment is also given by the area of the shear diagram, this can also be expressed by a shear distribution curve with a reduced area factor relative to the base shear. Several phases of the sharing of the load and shear diagrams can be developed and the best phase for the estimated elastic or post elastic behavior can be readily chosen. From the chosen phase the total building drift can be estimated. The effective period of the building can then be approximated, and for seismic design, a revised response base shear can be determined. This process can be quickly cycled to obtain a reasonable performance estimate. The load diagram for the shear wall gradually changes from a triangle with its base at the top to a triangle with its base at the bottom, and then to first order, second order, etc. parabolas with their bases at the bottom. This can be recognized as gradually lowering the c. g. of the shear wall load diagram. The load at the bottom comes from the frames which pick up load at the top but return it to the shear walls at the base. The load on the frames is approximated by an "X" type loading with positive load at the top and negative load at the bottom.

With this type of analysis it can be seen that the ductility of the back - up frame can be very important.

CONCRETE FLOOR AND ROOF DIAPHRAGMS

The subject of concrete floor and roof diaphragms has not received much attention, in testing or in the codes. The UBC codes from 1961 to 1973 required a diaphragm design force coefficient (C_p) of $KCW \geq .10$. Except for very short period shear wall buildings without a complete vertical load frame, the minimum C_p of .10 governed. For buildings with a T of 1 and a K of 1.33, the C_p value reached its maximum of .133. The 1976 UBC requires a diaphragm design force equal to the force at that level, obtained from the code distribution of the code base shear force. However, the required design force is not less than $.12\ w_x$, where w_x is the weight at the diaphragm level. For a building with the same weight at each level, this criteria can be expressed by the formula $C_p = K_1CW_x \geq .12W_x$. In this formula K_1 is a factor depending on the number of stories (if the story heights are equal) and C is the building base shear coefficient. The factor for the top level increases in the progression 1.0, 1.33, 1.50, 1.60, 1.65, 1.67, 1.75, 1.75, 1.75, 1.82, 1.80 as the number of stories increases from 1 to 10. The factor for the top level approaches 2 if only the code triangular lateral load distribution is considered, and a little more than 2 when a concentrated lateral load (F_t) at the top level is considered.

For low buildings, the 1976 UBC has considerably increased the force at the roof level diaphragm, above that required by the preceeding codes. Consider buildings with a frame factor (K) of 1, a site factor (S) of 1.5, and the same weight at each level. For buildings with a period of .5 second or less, the 1976 UBC design coefficient for the top level diaphragm would be .140, .186, .210, and .224 for 1, 2, 3, and 4 story buildings. The increase in force coefficient with height is the reflection of the first mode deflection shape and is therefore basically correct. However, the values do not give diaphragm coefficients which are consistent with the base shear coefficient. For a shear beam, the (MDF) multi-degree-of-freedom first mode contribution to base shear is only 81% of the base shear of a (SDF) single-degree- of -freedom system with the same period. The displacement and top acceleration of the same MDF system, however, are 127% of that of a SDF system of the same period. In other words, the amplitude of the displacement curve for the first mode of the given MDF system is 127% of that of the same SDF system, though the base shear contributed by the first mode of the MDF system (in effect, the mass acting with the first mode) is only 81% of that of the SDF system. Therefore, in relating the diaphragm level acceleration coefficient (C_p) directly to the base shear coefficient, the accelerations corresponding to that base shear seem to be underestimated by a factor of 1.27/.81 or 1.56. The contributions of the higher modes will add to this acceleration, though these high frequency vibrations may have little response effect on real structures. All of the above assumes a rigid diaphragm which responds directly to the acceleration of the vertical frame at the level of the diaphragm. This is usually essentially true for concrete diaphragms. It seems that we should look further into diaphragm lateral force levels.

The strength of a concrete diaphragm should relate quite directly to the strength of concrete walls, and yet the UBC has not applied the same load factor for shear nor has the same minimum reinforcing requirement been generally applied. Most concrete diaphragms have covered relatively small areas, however, and for poured in place complete concrete systems, the normal stress and temperature reinforcing has been adequate for shears. For topping slabs used as diaphragms, the problem is different.

Consider first the standard 2 1/2" or 3 1/4" concrete topping poured over steel deck. Tests for this combination have treated the deck as the diaphragm, with sheets interconnected by welding, and with the concrete as a stiffener. The side and end shears are transferred from the deck to steel edge members by welding, and the load is transferred from the steel edge members to vertical shear-resisting elements. The capacity of this type of diaphragm is extremely limited as compared to seismic demands and its use has been limited to small diaphragms, or where the designer does not really check diaphragm shear demands. For most floor diaphragms, the topping must be treated as the diaphargm if the code shears are to be resisted. The shear is then transferred directly from the topping to the vertical shear-resisting elements. Usually, the topping has had only a light mesh for arbitrary reinforcing, so those diaphragms depend almost entirely on the shear strength of the concrete, with little or no ductility The use of a light mesh is questionable on several counts. First, it does not comply with the minimum temperature reinforcing requirement which has been also depended on as a minimum seismic reinforcing. Second, the entire use of mesh shear reinforcing needs to be investigated. Light mesh is made up of cold drawn wire which has little of the ductiltiy that hot rolled small bars have. The welding of the mesh may also reduce the ductility. Light mesh has a questionalbe capacity to span shrinkage cracks and to provide any dowel action. If the shear friction concept is applicable, the reinforcing should be able to take the entire diaphragm shear. The friction factor (K_1) in the formula $U = K_1 \, pf_y$ should probably be taken as 1.0 for diaphragms because of the opening gap in shrinkage cracks.

The shear problems of the topping slab over steel deck are somewhat limited by the comparatively light weight of the steel frame and steel deck and the general use of lightweight concrete topping. However, it should be borne in mind that these diaphragms can be very large in low buildings and can be severely cut up by elevator banks, duct shafts, and stairs in multi-story buildings. A great number of department stores have been built in the last decade and most of these have very large floor areas with no interior seismic resisting shear walls or frames. The entire seismic lateral resistance has to be carried by the floor and roof diaphragms to exterior shear walls which are as much as 430' feet apart. Considering that a football field is 300' feet long, the span and loads for these diaphragms is great. Many times large openings are cut into diaphragm at the exterior walls, considerably reducing the length available for the shear transfer from the diaphragm to the wall. It can be seen that the performance of the diaphragms and their shear transfer details is very important.

A typical multi-story office building floor diaphragm is on the order of 100 feet wide by 200 feet long. Perimeter frames often take all of the seismic shear forces from the floor diaphragms. This does not create a serious problem with regard to the inertia forces from the floor and partition weight, even with the large openings in the central area which are typical of high rise buildings. But there may be a problem in providing lateral support to columns around the elevator openings. If a drift of as much as 1% occurs during a severe ground shaking, then the diaphragm must support the lateral forces of 1% of the vertical loads on their columns.

A much more severe problem occurs for topping slab diaphragms placed over precast concrete planks and precast concrete framing. This system will have a weight of about twice that of the steel framed, steel deck system so the lateral loads will be about twice as high. This means that the shear transfer to shear walls is twice as high per foot of shear wall as with the lighter system. Since this shear transfer is a problem for most diaphragms, it is especially difficult for the precast system. In addition, since the planks are not joined along their sides except by the topping, there is a potential crack at each plank joint. A typical system has a 3" topping over an 8" thick plank so there is a partially cut control joint 8/11 of the way through the system. There is a question as to whether these potential cracks, and all pour joints, should be treated as cracks requiring shear friction reinforcing. The integrity of the topping diaphragm and its connection to the shear walls is critically important since the typical precast concrete beam and column system has no reserve lateral capacity to provide against total collapse. This entire system needs research to insure its safety when designed to code forces.

The introduction of the shear-friction concept opens up a whole area of questions. The transfer of concrete shear across cracks or joints has always been a problem which has plagued the designer. Little or no research or code guidance has been provided. Designers have tried to use the unexplained low code bolt shear values for dowels, placed dowels diagonally to take direct stress, or have used keys, with or without counting on dowel help. The shear across the base of shear keys has not been covered by the codes, which have tended to treat all concrete shear as a diagonal tension problem. The introduction of the shear-friction concept, though based on a limited range of tests and not directed to diaphragm or wall problems, has therefore been attractive to designers as a solution to those problems. This concept may explain away a lot of problems with concrete cracking and joints, tight and open, but it seems to need a lot more research. Of first order in this research should be the transfer of shear forces from diaphragms to walls, concrete and masonry. There are specific problems of anchoring dowels into walls, particularly masonry walls, to develop the full tension capacity of the dowels. The actual shear value of the dowels should be tested, if it can be separated from the shear-friction action. The connection of diaphragms to walls is critical to the seismic performance of a building and adequate dowels are not expensive to provide so we should take a conservative approach to their capacity. However, the practical limits of application should be kept in mind. How big a dowel is too big to be adequately anchored in the wall? How close a spacing is too close to be effective? There are certain limitations to spacing which are based on such things as the core spacing in block walls. We really need dependable test data and it should not be too hard to obtain. In the same vein, we need tests to establish realistic shear values for bolts in concrete. The disparity between the values for weld stud values and bolts needs reducing or explaining. The relation between bolt shear, and dowel shear in shear-friction or composite construction needs explaining.

CONCRETE MOMENT FRAME SEISMIC FORCE RESISTING SYSTEMS

Seismic codes, up to and including the 1958 UBC, provided a seismic force formula which did not differentiate between seismic resistance systems. The seismic load was treated the same as other loads, using the same working stresses for design, except with a one third increase. This certainly seemed to imply, and it was thus inferred by many engineers, that seismic forces given by the code force formula were adequately provided for with even less than the usual allowable stress safety factors. However, the discussions preceeding the setting aside of the 13 story building height limit for Los Angeles in 1959 showed considerable concern about some aspects of this approach. There was general agreement amongst those who had looked deeper into the seismic problem that tall buildings, in order to survive strong earthquakes, had to have the ductility necessary to allow not only design stresses, but also yield stresses to be exceeded without failure. It was believed that buildings with steel frames had proved this capacity in past earthquakes. However, concrete frames designed by the usual procedures were suspected of being brittle and were therefore prohibited as a seismic lateral force resisting system in buildings over 13 stories high. The door was left open for the use of concrete frames if the required ductility could be proven, though no one knew exactly how much ductility was needed.

When the first SEAOC requirements were adopted by the 1961 UBC, the same general philosophy of design was carried forward. However, this code provided a direct link between building period of vibration and seismic force and it also provided a frame factor modifying the force formula. The frame factor for all moment resisting seismic frames was set at .67. Both the period and frame modifiers reduced the design lateral force for this system as compared to shear wall systems. However, the Los Angeles code provision prohibiting the use of concrete frames in buildings over 13 stories (or 160') high was included. This said, in effect, that concrete frames as seismic force resisting systems were too suspect to be used in buildings over 13 stories high, but were flexible enough to be designed to lower than average seismic force levels in other buildings. This apparent inconsistancy was simply the result of hanging on to empirical judgment for the major part of building construction and yielding to theoretical force predictions for the little tested tall buildings.

In order to qualify concrete frames for buildings over 13 stories high, the concrete industry undertook the task of determining what it takes to make a concrete frame adequately ductile for seismic design. A few specially reinforced concrete frames were tested to prove that concrete frames could be made adequately ductile, but the key work defining the needed provisions to obtain ductile concrete frames was the PCA sponsored earthquake design book. In chapter 5 of this book, Professor Newmark provided an analysis of concrete tests into the inelastic range. In chapter 6, suggestions were made for the design and detailing of concrete frames to provide for ductile yielding. These two chapters were used by code committees as a basis for Section 2630 of the 1967 UBC, which defined a ductile concrete frame which could be used in buildings over 13 stories high.

Some of the requirements of Section 2630 were so complicated, so difficult to design to, so restrictive, and so difficult to apply in the field that no buildings were built exactly to this standard during the next few years. For the few buildings over 13 stories high which were required to be designed to this section, the building departments involved had to make some liberal code interpretations to allow a feasible design. Some buildings under 13 stories high were designed to comply with their engineers own interpretation of essential ductility requirements. However, since the code required no special ductility provisions for these low buildings, non-ductile concrete frames continued to be built. The need for ductility was now fairly well recognized, but the need for some of the almost prohibitive provisions of Section 2630 was not generally accepted. After long debates, caused mainly by a shortage of the test data needed to settle the debates, a somewhat modified and relaxed Section 2630 was included in the 1973 UBC and required to be applied to all concrete frames. Today there are still many provisions which severely limit the use of concrete frames as seismic resisting elements and which are not well supported by test data. There is also debate about how to comply with some of the provisions.

Chapter 5 of the BNC book looks first at the compressive stress strain relation for concrete cylinders and finds that significant strains are reached before failure. The strength in direct compression starts to degrade after a strain of .2% but the degradation is not present for strains as high as .7% or more in flexural compression because the compression block changes shape with local yielding. The very great increase in compressive strength and useful strain in compression cylinders under high confining fluid pressures is then examined. A formula for the effect of fluid pressure on compressive strength shows that the compressive strength is increased by 4.1 times the confining pressure. It is shown that the benefits of fluid confining pressure can also be attained by confinement reinforcing. A formula for computing the confinement pressure of spiral reinforcing at concrete compressive yield stresses was developed. It is stated that tests indicate that "rectangular hoops may have a reduced efficiency of as much as 50%". On the basis of this statement and the formulas provided, it can be shown that the UBC minimum confinement reinforcing provides an estimated 25% increase in compressive strength of the core area. The estimated confinement pressure at yield of the hoop steel is only equal to .06 f'_c, or 300 psi for 5,000 psi concrete, but this is expected to allow the development of compressive strains in excess of 1% without failure.

The rest of chapter 5 deals with ductility in bending and combined bending, and the shear provisions necessary to assure ductile bending. Most of the flexural ductility discussion involves the same considerations which have been recognized in the development of ultimate design concepts for concrete flexural members. The principal consideration is the assurance that the beam will yield in flexure by yielding the tension steel before the concrete ultimate compression stress is reached. It is shown that concrete confinement will increase the compressive strength and ductility, but the key is still to limit the amount of tension reinforcement. Considerable ductility is provided if shear failure is prevented and flexural tension yielding is assured even without confinement of flexural compression.

Chapter 6 provides a discussion of needed design considerations for concrete ductile frame. Four important points in the design to achieve ductility are listed:

1. Use of transverse or shear reinforcement to make the strength in shear greater than the ultimate strength in flexure.

2. Limitations on the amount of tensile reinforcement, or the use of compression reinforcement, to increase energy-absorbing capacity.

3. Use of confinement by hoops or spirals at critical sections of stress concentration, such as column-girder connections, to increase the ductility of columns under combined axial load and bending.

4. Special attention to details, such as splices in reinforcement and the avoidance of planes of weakness that might be caused by bending or terminating all bars at the same section.

Section 2630 of the 1976 UBC provided for these four points as follows:

1. The design ultimate shear capacity was required to be equal to the shear capacity which may be induced by the sum of the beam moment capacities computed on the basis of the specified material stresses.

2. The area of flexural tension steel was limited to insure tension yield before compression failure.

3. Special transverse reinforcement was provided at each end of columns, essentially as suggested in chapter 6 of the BNC book.

4. Special requirements, such as splice locations, were imposed for reinforcing details.

The 1973 UBC contained revisions to Section 2630 directed toward points 1 and 3.

1. The original requirement provided shear capacity for the specified yield strength of the reinforcing, which is a minimum strength specification, not a maximum. The ϕ factor reduction for moment capacity was also included by implication. The new provision requires the moment capacity to be computed for a steel strength 125% of specified yield, and without a ϕ factor reduction.

2. Providing rectangular hoop ties with a volume twice that of the regular ACI spiral requirement, and with a limit of 25% help from cross ties proved to be generally impractical. The required ties were either so large that they could not be bent to a tight rectangular shape, or they were so closely spaced that concrete could not be placed properly. It must be recognized that spirals do not have either the sharp bend problem or the splice lap and hook problem. A volume of hoop ties twice that of spirals is a lot of transverse reinforcement, particularly in small columns where the required spiral volume is high. The need for this BNC recommendation was therefore re-examined. First, the ACI formula for the required volume of spiral does not really relate to the ductility problem, except that columns so designed have proven ductile. The formula is designed to provide additional capacity to the core concrete to make up for the loss of the concrete outside the core under ultimate loading. The volume of spiral reinforcement is thus related to the ratio between the core area and

the gross column area. Small columns therefore require a large volume of spiral and large columns require a small volume. A reduction in ϕ factor provides this margin of capacity for axial loads for tied columns. It seems that the formula for spiral reinforcement should not apply and the same volume ratio for hoop tie confinement should apply to all columns. A reduction in hoop tie requirements was also indicated by the fact that none of the test frames had used the Section 2630 tie requirements and all had performed satisfactorily. The 1973 UBC revision reduced the rectangular column confinement tie requirement based on the spiral column formula to a factor of 1.33 instead of 2 times the spiral formula. Cross ties may be used as needed and are required to be places at not over 14" apart in plan dimension. This has made the confinement tie requirement practical, but the question still arises as to whether the spiral formula relates at all.

A problem much more difficult to solve than the requirement for confinement reinforcing in columns, is the need for confinement and shear reinforcing in the joints. The shear forces are greatly amplified in the joints and the practical difficulties of providing large amounts of reinforcing in the joints are great. The proper analysis of joints, for diagonal tension, diagonal compression, and confinement are still being investigated.

Section 2630 places many restrictions on concrete frame design which were not suggested by the BNC book. The dimensions of columns and beams are restricted to near the dimensions of the few test frames which have been tested. More restrictive than this, however, is the requirement that the bending capacity of the columns exceed that of the beams. Yield bending of columns, which is already heavily protected by the column confinement requirements, is then prohibited. The effect of these requirements is to reduce the use of concrete frames to a very small percentage of buildings. Basically, the frames must be used the same as steel frames, as a structural system which has architectural cladding. This turns out to be a strictly economic alternate which seldom favors the concrete frame. Architects often want to have the architecture and structure expressed as one in the exterior building expression. Under the present UBC code, this can only be if the architect wants to see a concrete frame of limited frame dimensions.

If all of the code limits are necessary to insure a safe building then they are justified. But the need of many of these arbitrary provisions is far from proven. The 1971 earthquake experience of the Olive View Hospital illustrates this. Tied columns weak in shear failed very predictably, causing almost all of the damage. The spirally reinforced columns not only proved their ductile value but also proved that bending yield in confined columns does not destroy their axial load capacity. These columns were subjected to lateral loads and distortions well beyond anything anticipated under our newest codes. The beams framing into these columns did not conform to any of the ductile frame columns, and yet they caused no collapse. Two essential features kept the hospital building from collapsing, elements which did not fail in shear, and columns with spiral confinement. We come back to the four essentials for concrete frame ductility enumerated in the BNC book. It seems that many legitimate forms of concrete frames are being eliminated by the code with a degree of caution not at all consistent with the general treatment of seismic design. Tests of many frame configurations could be of great benefit, not only for new construction, but also to seggregate the real risks in existing buildings.

Two areas of concrete frame design which could use research are the combination of P/S and conventional reinforcing in concrete frames and the real shear capacity of concrete columns.

A problem with conventional reinforced concrete frames (as opposed to steel frames) is that there is no option for the fixity of the joints. Vertical load beam end moments are always involved, whether they are needed to support vertical loads or not. These moments, added to seismic lateral force frame moments, complicate and handicap seismic frame design. A prestressed floor system, designed to balance dead load moments, provides joints which are relatively unstressed in flexure, except when they are subjected to seismic forces. Conventional frame reinforcing can therefore be provided to resist just the flexure due to seismic forces. This system has merit but not enough research to gain general acceptance.

The shear capacity of columns subjected to axial loads and concrete creep and shrinkage depends on many factors, including the time factor. It may never be possible to predict this capacity closely for any time factor but the importance of column shear capacity warrants more research than has been directed specifically to columns.

MISCELLANEOUS TOPICS

Bolt Shear Values and Concrete Bearing Stresses

Code allowable bolt shear values and bearing stresses have historically been set at such low levels that they have always handicapped and confused designers. In many cases the use of these low values leads to what seem to be unrealistic design details and the basis for the values has been a puzzle to most engineers. New values based on research are needed.

Concrete Mixes for Congested Reinforcing

Since the early days of concrete mix design the value of as large size aggregate as can be readily placed has been stressed. The use of as dry a mix as can be placed has also been stressed. Efforts to comply with these principles has led to many job problems, with the congestion of reinforcing which is often required for seismic design. However, it has been found that one half inch or smaller aggregate are needed to insure concrete strength with lightweight aggregate, and it is indicated that these small-aggregate mixes require no significant increase in cement for a given strength. The addition of a small amount of water makes the mixes much easier to place properly and seems to require only a small amount of additional cement, which does not significantly add to shrinkage. It seems that this whole subject needs enough research to better advise engineers regarding concrete specifications.

Detailing Reinforcing

The detailing of reinforcing, particularly for concrete ductile frames, is very important to construction economy and seismic performance. Research aimed at providing standard details acceptable to engineers, fabricators, and contractors, is needed to improve seismic design and construction of concrete systems. For instance, the limitation of round column reinforcing patterns to 6 and 12 bar patterns is feasible and eliminates the interference between horizontal reinforcing and column reinforcing common with the random numbers of column bars. Square spirals would solve many of the ductile column tie problems but neither the fabricator or designer wants to go out on a limb with this detail. Efficient use of bundle bars in columns needs research. If impartial practical research could establish more standards for the detailing of seismic reinforcing, everybody would benefit.

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED
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AN OVERVIEW OF THE STATE-OF-THE PRACTICE AND OF USER NEEDS FOR IMPROVING ERCBC

CANADIAN ASPECTS

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PREFACE

When rereading his own notes, this author found them to project a rather dim view of past performance in ERCBC. As well, the degree of present understanding, from basic principles to practical everyday application, appeared to be rather minute when compared to the task. This rather discouraging image of the situation at the designer's end is perhaps the result of the author's relative ignorance as well as a feeling of lost comfort that one previously had when everything seemed to be rationally clear. Earthquake resistant design was far removed geographically or in terms of scientific endeavor and its requirements where a matter of simple application of an algorythm laid down step by step in the building code if earthquake was altogether considered worthy of design measures and seismic effects were not simply relayed to the category of "acts of God" one does not even try to do something against.

If the commentary appears to sound a little irksome, this author must ask forgiveness for relating a situation of uneasiness and confusion. It appeared to be more useful to report on the state of the art including the shortcomings, rather than to convey a polished picture of deceptive success. Successes in ERCBC would be hard to prove in any case where the time of test is not in our control. The conscience of most designers has been awakened to the fact that in ERCBC there is something beyond blind application of a simple algorythm. This something, however, is quite out of reach to date for a designer to practically apply.

In this sense perhaps the development can be seen as quite positive; as the consciousness of possible and necessary improvement develops, so does the feeling of confusion about things yet unknown, and the present would only be the steep part of the way "per aspera ad astra".

Designers have designed and builders have built earthquake resistant structures for more than a decade to the standards of the day, only to learn subsequently that the standards had changed and the structure might just not be as earthquake resistant as one thought it would, or the other way round as the case may be. This has had a somewhat disquieting effect which this author could not help but let penetrate in this commentary.

INTRODUCTION

The call for eathquake resistant design has been with the profession for some time. It is forcefully expressed in Building Codes which are equivalent to laws in most cases and must therefore be followed in letter and intent.

Besides their function as legal documents Building Codes are also sources of design information, sometimes even the only such sources available to the designer, as in the case of ERCBC.

For the following commentary, the National Building Code of Canada, 1975 edition, shall form the basic reference as a good example of a modern Code. The author confesses to the knowledge of some papers on ERCBC most of which, however, do not provide design information directly but are commentaries themselves on portions of building Codes, or on particular aspects, and the Code still forms the most complete and comprehensive sediment of knowledge in the field, as well as the only one within reach of every designer.

The double function of Building Codes results in some problems for the designer, in particular when dealing with ERCBC. On the one hand he is tied to the rules spelled out in the Code. On the other hand the Code when followed to the letter, does not provide discharge from the designer's duties toward the law. Recent court practice is reported to tend towards blaming the designer anyway for a mishap, whether or not he complied with the code. This is a consequence of the high priority our system of jurisdiction puts on finding someone to blame and, maybe rightly so, the designer is considered the only one who can be made responsible for the consequences of a mishap, collapse or malfunction: if following the Code turns out not to be good enough, he should have known better.

Follows the question, how he could have known better, which in the case of ERCBC leads to a logical short circuit because no other sources of solid information exist within practical reach of the designer.

If one approaches the design problem naively, the Code still provides a set of quick kit rules for the faithful. Static equivalent loads are derived following a sequence of empirical formulas drastically symplifying physical reality, and these loads are then to be treated in the same fashion as any other load effects. Special premiums are offered if the design complies with another set of rules called ductility provisions. If one tries to rationalize these rules, it becomes quickly evident that this cannot be done because the relationship to physical reality is entirely empirical. Therefore, where gaps and inconsistencies exist, or questions arise as to ranges of validity, ways of application, or in cases that do not fit the classifications given along with the procedure, there is no recourse to the use of analogy, extrapolation or similar ways of extending the recipe. In recent literature also, the entire area of ERCBC seems to be very much in flux and solid indications to hang on to are hard to come by.

LOAD EVALUATION

Let us examine first the methods pertaining to the establishment of earthquake loading effects. When the faithful designer turns the pages, he will find that the National Building Code, in Commentary K provides an alternative method to the quick kit formulas which is based on the participation factor approach. This method gives the impression of being based on more in depth knowledge and less simplified notions of the analytical problem in question. It requires substantial work on substantial computers. Impressive amounts of figures are produced in the process which, by their mere quantity exert some persuasion as to the trustworthiness of the results. Joe Blow's quick formula however, has not left the mind of the designer and he does what one does with shortcut methods of this type, he uses them to check the results of the "better analysis".

And here it is where the puzzle starts to become manifest and confusion descends onto our designer. Results of shortcut formulas in structural design are expected to be within sensible limits of the "true" or "exact" ansers, say within ten percent or so. When this is not so, doubts on the validity of the methods used rise progressively with increasing margins of difference. In the case of recent earthquake design codification, these doubts have reached alarming levels, as Fig. 1 may illustrate. It represents a survey of some typical projects (see Table 1 for short description). Compared are the results of the quasistatic method and the "dynamic method" based on participation factors, both from the 1975 National Building Code of Canada. If discrepancies exist for unusual structures like No 15, which is the CN Tower in Toronto, this does not overly surprise or disquiet the designer because obviously any empirical formula has its limits of application. Where on the other hand, different methods yield considerable divergencies as shown in Fig. 1, for structures that are well within the description the methods were meant for, then the faith of the designer starts failing and the credibility of the Code reduces. As the designer is hired with the mandate of designing for economy, there is a very real compulsion to pick one's choice and apply whichever method causes the lesser "trouble" in terms of expensive gimmicks such as complicated reinforcing, extra stirrupts, and the general nuisance of having to consider yet another cluster of ill defined parameters. On the other side there is the sharp young engineer and City Hall who sits on the building permit. He is not at all confused yet, and wants the Code followed to the letter, whatever in his mind that letter amounts to. Due to this sort of conflict, design decisions then tend to become a matter of expediency and diplomacy, rather than rational deliberation.

Since the designer has no practical way of putting the several methods to the test only the wise man remains to turn to for help, perhaps in the person of the secretary of the Code Committee. Secretaries and other wise men though have had a habit of being rather taciturn when it came to the kind of questions that smell of sharing in responsibility, and the designer then finds himself facing a task he is not really up to.

The engineer is required to design buildings economically as well as to the standards the authorities impose, in letter and intent, and, beyond those standards, to resist in satisfactory manner all loads or effects that may occur. This situation is quite precarious when instead of a consistent theory, only empirical methods exist that yield results in a wide scatter.

Again, the temptation is great to take whichever method gives the lesser trouble in terms of design effort, materials, and difficult explanations to clients or City Hall. Earthquakes seem to happen only in newspapers, sufficiently far removed in time and space, and who is going to prosecute an engineer when a major disaster has struck... This then amounts to a dismissal of earthquake resistant design as something of lesser importance, following the tested principle that something so complicated cannot be important, and a minimalistic attitude is the result of it all.

If this author is allowed a shot at the probable reasons for the disturbing inconsistencies that exist among the various methods to determine earthquake loads, it appears that, as ever so often, a problem is being attacked with the wrong means, the means having been selected on the basis of impressive looks rather than real qualification for the task. Like the general who shoots with a 32 inch gun on guerillas scattered in the mountains, we have adopted the elastic theory as our weapon of choice, and we blast away with ever bigger and more sophisticated computer programs on a problem that lies somewhere out of reach, namely beyond the applicability of the elastic theory.

If the effects of earthquakes are to be absorbed by a building in a plastified state, the analysis should conceivably start with what we know about that state, and the elastic theory be reserved to cases where it applies.

For the great majority of structures it is assumed and admitted today that they will indeed leave the elastic state in the course of an earthquake with probable intensity and absorb energy and movements dictated mainly by the ground motion directly rather than via the stiffness properties of the structure. Should not then out design philosophy be centered around this fact rather than the elastic model we can not afford to let apply ? From this angle it would appear that methods like the "participation factor approach" to the determination of a loading pattern are of very little value. On the contrary they tend to project a deceptive image of scientific rationality, covering the poor state of our knowledge of the real problem. Perhaps then, the quick kit static approach is still as good as any method and, being composed of honest fudge factors, a tool more handy and transparent than pretentious elactic methods.

Indications seem to exist that the best strategy to deal with the problem may lie in an entirely different direction than the determination of loads to be resisted, and to base rules and thinking on deformations, motions and energy rather than forces.

Certainly it will be difficult for the mind of the engineer who was educated in the traditional way, to leave the path of thought that led him safely from A to Z in all his previous design tasks. It always began with the setting up of loads, it proceeded by providing a structure resisting these loads and ended by providing proof through analysis that this resistance would occur in a satisfactory manner, without undue side-effects impairing the serviceability and safety of the structure. In most cases, it was expedient and rational to select loads as the principal terms for expressing such design criteria.

In the case of ERCBC though,this traditional way of comparing loads with resistance would seem not to have been successful for expressing criteria of structural behaviour. No satisfactory and convincing method exists to determine those loads, and another important fact can be observed quite readily from Fig. 2.

In the inelastic range, the state of a structure is not described well by information about loading levels. If the hatched band in Fig. 2. is to suggest typical information about loading levels, including uncertainties, it becomes clear that a wide range of states of the structure exists that are all possible as far as our typical knowledge about loading is concerned. In terms of safety, stability and serviceability however, these states vary over a wide range, from buildings that show hardly any damage over to those where collapse through loss of stability is imminent. Therefore loads do not appear to be a useful way to express criteria for building behaviour, once the structure is reacting principally in a plastic manner. Perhaps then, ERCBC should center around other terms describing building vehaviour, such as deformations and their consequences, or absorption of energy.

A tendency in this direction can be observed in recent editions of building codes where rules are accumulating that deal with the effects of large deformations, or with the limitations of such. Progress in this field seems to be quite recent and still very much in motion; therefore many open questions exist, some of which shall be examined in the following.

DESIGN PRINCIPLES

If, for want of a satisfactory analytical approach to determine building behaviour in the traditional manner, other methods have to be found to achieve the goal of earthquake resisting structures, a new set of principles must be conceived as a basis for design. The basic terms of this seem to emerge in the form of such concepts as ductility, after shock stability, second line of defense, or the like.

In the following paragraphs, a number of considerations shall be examined, relating to those new principles, and from the point of view of the designer. He who has to accept without practical recourse what he is fed in the building codes, presently has to make do with a collection of rules related to the principle of ductility and substituting its application. These rules appear to have been produced in a great hurry, implementing research information freshly brewed from the lab, quite unlike some other design novelties such as f.i. ultimate strength design, the principles of which were very well known for quite some time prior to its implementation in official design documents. Inconsistencies, unanswered questions and missing links therefore exist in the somewhat confusing collection of regulations to be followed, and in some applications the rules become outright restrictive, drastically limiting the design options that existed before.

The tendency to have requirements included in codes before consistent knowledge exists to support them, has been a disquieting trend in recent years and ERCBC is not the only area where it happened. Another example of similar importance is the field of design against progressive collapse where the Code Committee saw fit to include a full-fledged requirement in the Code to this effect before any notion was available to the designers on ways to do this. It has caused considerable frustration to the designers that they have to follow blindly a set of rules the basis of which they have not been explained. Since the Code has the force of law, it is a risky game to wave it even where it obviously does not make sense. In other areas, considerable gaps exist in the variation of cases covered by the rules which cannot be bridged rationally as no rationale exists among the different regulations. The designer is then left to his own judgement which at this time he has had very little opportunity to educate.

Ductility

The principle of ductility seems to become the centrepiece of ERCBC. In search for a clear definition of this basic term, this author has spent considerable time and effort, without much success. The exercise was rather frustrating and turned up all sorts of "ductilities" depending on the particular application, or the individual author. An unequivocal definition, however, was not to be had ... In spite of this, ductility appears to be the main pillar of faith in the field - a ductile pillar indeed in various respects.

Every structure, beyond the simplest single element arrangement, exhibits a characteristic behaviour at high loads similar to Fig. 2. In most cases, yielding or other energy absorbing processes start gradually while most of the structure still remains elastic. The original stiffness of reinforced concrete structures in particular, will vary substantially, yet remaining within the limits of purely elastic behaviour, due to the lack of tensile strength of the concrete and consequent opening of cracks.

All characteristics exhibit a definite level of maximum load acceptance followed by a more or less abrupt decline, due to loss of strength or stability or both. What exactly does ductility mean when applied and demostrated on this typical diagramme and which is really the point of failure against which safeguards have to be measured ? Is it the point of maximum load acceptance, or a state somewhere beyond, perhaps where the structure will become incapable of bearing loads due to its own weight, and/or subsequent normal events such as wind or a lesser shock. Answers to this sort of basic question are urgently needed.

Inelastic Motion

Dynamics of inelastic structures is a field where knowledge seems to be virtually non existent, in spite of the fact that we design buildings for this very range of behaviour. In present building codes the gap in knowledge is bridged by relating the supposed reality to the easier-to-analyse model of elastic dynamics, via ductility. Ductility factors describe a maximum range of deformations the building can supposedly tolerate without collapsing, provided the deformations occur in a pattern resembling the elastic configuration.

When it comes to predictions about effective deformations during and after an earthquake, no information or methods exist at all and building design is strictly limited to the only criterion of collapse. Other objectives cannot be formulated presently in the realm of inelastic dynamics. This means for the great majority of structures that we do not effectively know what will happen to them with the only exception of an overall collapse that we hope to have prevented.

If one looks at likely mechanisms in a structure responding with large deformations to ground motions, it can be seen that the configurations of the deformed building do not necessarily correspond to the elastic model. Therefore, energy distribution as well as the stability conditions can be dramatically different, and the assessment of safety which was based on a ductility type of relationship, appears rather questionable.

One example for this is illustrated in Fig. 3, where a typical building frame is subject to ground motions. The most likely mechanism in a good design is a beam mechanism with only a minimum of hinges in the columns which are probably concentrated near the base. The column hinges will result in large local deformations in the bottom storey, dramatically changing the excentricity on that particular column section. Local instability may occur without the rest of the building having deformed much, partially due to the properties of typical characteristics for beams and columns. The example in effect represents a case of very low system ductility, in spite of careful ductile detailing and a mechanism that started off looking quite good. It cannot be proved that a particular building will react in this fashion but it cannot be disproved either. Perhaps the illustration relates to a very construed case but certainly one that is conforming to the code category of ductile structures and is therefore equipped with the lowest strength reserves while earthquake design loads were reduced by the maximum rebate.

Large Deformations

In buildings of high ductility it is assumed that considerable nonlinear deformations will take place to absorb movements beyond the ones the building will accept elastically. These large deformations themselves have not, until recently, been made a subject of consideration. Obviously, after having been subjected to a major shock, a ductile building will not return to its original state of stiffness and deformation. Large internal stresses will be present and energy absorption characteristics will have changed. Also, on slender structures, the stability situation may have been affected. Analysis of even relatively simple cases is out of reach for any such consideration, even for a deterministic time history of a particular response, to say nothing of probabilistic evaluation.

Theoretically the particular configuration of a deflected building in which it is left by an earthquake may not be very important, in practice this is very much the case, because it will decide f. i. whether the building shall be salvaged and repaired or that it must be condemned. Therefore the limitation of permanent deflections becomes an interesting objective. No clue exists as to methods for achieving this except that the Code discourages the design of stiff structures by means of penalizing them with heavier earthquake loads.

Stability

Another question arises around the second order (P- Δ) effect. The intent of Codes is obviously that it be accounted for, in spite of the fact that statements to that effect have been placed rather inconspicuously or not at all.

The P- Δ effect can be read off a characteristic diagramme like Fig. 2 in different manner. For instance in the elastic region it can be seen as an increase of actual deformations over those caused by the lateral loads solely. In the inelastic range this is not possible and the P- Δ effect can only be measured in terms of effective reduction of the maximum resistance from that which would be offered to lateral loads acting alone. Which one of the two should now be introduced ? Or is it as this author suspects, admittedly based on rather primitive visualisations, that a "dynamic" P- Δ effect applying to rapid shaking movements is yet another thing ? The P- Δ effect is normally evaluated elastically and while it amounts to only 5 to 10 percent for reasonably stiff buildings, it is carried as a general correction and accounted for in overall loading considerations - even this is not being done in every cases to this author's knowledge, leaving it to the safety factors to take care of it. For stubby and stiff buildings, that may not be of much consequence, however ductility and the "soft way" is being preached to us and inevitably, this will be reflected in the general attitude of designers. We will see a class of engineers who do not believe in strength as much as their fathers did but who are accustomed to a "judo" type thinking which may conceivably get them in trouble with the wind, the P- Δ effect, and the unfortunate tenants of the top floors.

To summarize once more the feeling of the practising designer towards ERCBC, it seems that he is required to utilize methods and rules that reach him fresh from the kitchen, in a half-cooked state, as opposed to the solid ends he is meant to achieve, namely "to make buildings safe". To determine what exactly "safe" means and just how safe buildings should be made, remains his own problem. We appear to be reverting, for better or worse, to Hammurabi's law and presently the only high probability that can be safely stated is, that many design engineers will have to be put in jail subsequent to a substantial earthquake. In order to lower that probability, a substantial increase in knowledge and comprehension will be necessary on the part of the profession, and above all, a clear and consistent concept of the earthquake resistance of structures.

PRACTICAL QUESTIONS

The following is a list of specific questions that had to remain unanswered in the recent past, in connection with design cases most of which were subsequently built, with the problem having been resolved to "our best judgemen All comments are based only on ERCBC requirements as expressed by the National Building Code of Canada, 1975 edition, and CSA Standard A23.3, 1973.

1. The disagreement among the several methods for establishment of loads has already been mentioned as a principal source of doubt (Fig. 1).

2. The classification of structures in one of the K-factor categories has been a major point of discussion. The requirements for "ductile" structural elements are very restrictive and in some cases lead to impractical solutions. Therefore an incentive exists to circumvent these requirements in some manner, by using different methods of analysis or assumptions, "forgetting" brittle members etc. in order to make the structure eligible for the premium ductile categories. Lacking clarity in several respects facilitates this.

3. Ductility in general. When will it be possible for a designer to evaluate ductility, based on a clear definition of the term, and his knowledge of the structure, for instance in a form similar to Fig. 2. The ductility factor is presently contained in the K-factors of the Code recipe. It cannot be varied beyond or between the few categories that are stated in the Code.

4. The determination of the natural period of a structure is one of the ingredients of ERCBC. For the quasistatic method expressions are given to determine it and it is implied that other methods can be used. Usually these other methods, sometimes with dressed-up assumptions, can be made to produce much longer periods, hereby yielding a substantial rebate in earthquake loading. Since dynamic response to earthquakes is a matter of more than just the first mode of vibration, it would seem that its natural period is not really a parameter of such eminence; even more so where the building response is in the inelastic range anyway. If on the other hand this "natural" period only represents a symbolic empirical parameter, then it should be declared as such and the empirically determined factor should not be replaceable by a "true" natural period.

5. The significance of the building dimension D is not explained. It appears in various places in the algorythm of load determination. Why would buildings react differently because of exterior dimensions ?

6. Some questions concern the clarity of the rules for ductility classification :

   - The 25% resistance of the ductile frame, must it be 25% at the base or at every level, even when the frame, as is usually the case, does not receive any such loads elastically ?

   - The elastic load sharing between shear walls and frames, does it have any real meaning at all for ERCBC ?

   - What is a complete ductile moment resisting space frame. Can it include such elements as slabs, or columns integrated with walls ?

   - If a frame has to resist 25% of the lateral loads, how should this load be applied on the frame. In its elastic state, disregarding the shear wall, or in which other manner ?

   - What is the basis of the 25% rule ?

   - When there are shear wall-type elements in a building not capable of resisting the total load, but a frame that does so, how must these shaftwalls be reinforced ? And how much load can be assigned to them ?

7. In many cases the theoretical prevention of uplift at the foundation becomes a major problem for the design of shear wall/frame buildings. Is it really necessary to counteract completely a momentary uplift of the core, or what sort of criteria or philosophy should be applied.

Many people believe in the usefulness of rock anchors for this purpose, sometimes ignoring the limited durability of such structural elements. It is hard to visualize a building overturning due to uplift at the foundation in an earthquake, with shocks in both direction rapidly alternating.

8. A notion exists that buildings should be tied together at the foundation level. Why ?

9. The zoning of earthquake intensity appears to be very crude, and since the participation factor approach allows to base the loading assumption on ground acceleration figures rather than zones, it becomes a matter of geography which method is the more "economical". For instance at a place like Quebec City, a ground acceleration of 7.1 places it in zone 3 which also includes locations like Victoria with 11.1 or the notorious La Malbaie with 49.5. For the sharp designer then it pays to use the K factor approach for La Malbaie and Victoria, but the participation factor version for Quebec. This is obvious nonsense but it is not clear how the inconsistency should be resolved. Could perhaps the static method be adapted to ground accelerations where known instead of zones.

10. Very often buildings are combinations of different structural elements and elastic analysis, if it is at all capable of producing meaningful results, is of limited value. Many buildings with major elements of masonry, precast concrete, or timber, belong in this category. When there is no sensible elastic model of the structure, what methods should be applied to determine loads ?

11. The Code requires that buildings either be separated by a sufficient distance or that they be constructed integrally. This seems to rule out the use of conventional expansion joints, a restriction which is obviously impossible to apply because expansion joints are needed. A complete separation of two building portions would mean a gap of many inches at upper floors which will have to be bridged somehow. How ? Can this requirement be supported by real experience ?

12. Very often, existing structures are entering consideration that were designed to older less restrictive standards or to none at all. In the theoretical fulfilment of his duties, an engineer would have to condemn those structures as soon as he has been exposed to the knowledge of those facts, and refuse to be involved with them, unless they are fully equipped subsequently to comply with ERCBC. No criteria exist either to guide the designer in this decision, or, in case the existing structure can be assessed and improved, on ways and means to do this.

13. Ductility requirements are very complex and in many cases their relevance is anything but transparent. In some instances discrete classification replaces gradual adjustment of some parameters, in the same sense as the earthquake zones replacing the ground acceleration.

The following particular points have produced difficulties and doubts recently :

13.a The requirement for ductile shear walls to avoid brittle fracture of the section in some cases necessitates enormous amounts of reinforcing, as well as the incentive to reduce thickness and length of the walls, because the reinforcement to be provided is proportional to the section modulus. At the same time the requirement can be waved in the upper half of the building, if a plastic hinge is not expected to develop. Why exactly the upper half and what criteria are to be applied to establish the absence of an expected plastic hinge. Since the reinforcing in question can be quite substantial in terms of cost and nuisance, this becomes an important point, more so than it might seem philosophically.

The corollary requirement for column type ties around the concentrated reinforcing at the ends of the walls appears to have a less than solid basis. The concentrated reinforcement has been determined for a tensile ductility criterion. Why then should it be tied as in a zone of high compression even if that compression never occurs, as can be shown in some cases ?

The concentrated reinforcement is presently not restricted in terms of density. As there is some incentive to accumulate all of it at the very end of walls, the density should be limited, f.i. by relating it to similar restrictions for columns.

13.b For columns similar questions can be raised. Plastic hinges should if possible be kept away from columns, or so one is advised. In most cases this will be a result of high compression load in columns anyway and plastification occurs in the beam. Why then must all columns be equipped with the special confinement reinforcing near the joints, even if no plastic hinge will ever occur in these areas ? This confinement reinforcing has been found to be a major problem practically because it requires special persuasion as well as very extensive field checking from the engineer. It also makes for a substantial quantity of steel, as well as for increased complexity with placing the concrete. Is the severity of this requirement really based on solid factual ground, and does it have to be applied as generally as it is now spelled out ? Ridiculous cases are quite frequent where confinement reinforcing must be placed in massive main columns at every intersection with floor bleams, regardless of proportions.

13.c Slabs and sometimes walls resisting forces transverse to their centre plane are often the major elements resisting lateral loads. As they are generally considered members of the family of ductile building elements provided the degree of reinforcing is within certain limits, would this be applicable in ERCBC too, or are narrower limits to degree of reinforcing needed, or other restrictions to make them eligible as ductile structural elements ?

At this time, a flat slab building cannot be classified in terms with any method and opinions of considerable divergence exist among designers as to the value of flat slabs, as lateral load resisting elements. The same applied for walls or wall columns.

13.d The distinction between a column and a wall becomes important in ERCBC. It is not given anywhere, and no limitation or catalog of acceptable columns shapes exists. In practice, many column sections occur;among them are oblong, L, T or Z shapes, often in connection with flat plates. At this time the designer is left more or less to his own taste in declaring as wall or column whatever he pleases with all the consequences in reinforcing requirements. Again expediency or other considerations foreign to ERCBC will then decide on the design features where better advice is lacking.

14. Prestressed or posttensioned structures are built everywhere in the world, and therefore they will also occur in areas of high earthquake risk. No rules, however, seem to exist to determine or to design for ductility; and in this case in particular, the designer's own conjectures will not yield any clue as to what may be possible, advisable or required. This author has to confess to the rejection of a posttensioning alternative on a particular project, one of the principal reasons being that he was unable to establish any notion relating this method of construction to ERCBC, in order to live up to the intent of the Code. It can surely not be the intent of a Code to rule out an entire method of construction such as posttensioning. What then is the state of the art in this respect ?

15. In some cases difficulty exists with establishing the effective ground level, i.e. the level at which the resultant earthquake shear is imposed on the building. Which criteria should be applied ?

16. Vertical accelerations are reported to occur in connection with earthquakes. The Code does not make any reference to these. What is known on the amounts and effects of vertical accelerations and what provisions if any are to be designed for ?

17. Infill elements are usually disregarded in structural analysis. Sometimes this includes blockwalls, partitions of various types, or facades

which are of considerable stiffness and sometimes even strength and/or ductility. In photographs of post-seismic conditions it can be observed that many of these elements are still in place, sometimes not even damaged. What is the philosophy about these elements ? Is it conservative to assume that all brittle elements will be knocked out and out of order, or can they have deleterious effects on the load-carriing structure, f.i. the ductility of a frame ?

18. Are there any rotational components in seismic ground motion, and if so, in which sense. What are the consequences ?

19. What has happened to the "soft storey design" that was proposed some years ago ? Are there any definitive arguments for or against it ?

CONCLUSION

Earthquake resistant design appears to have reached a state of stagnation where no real progress is being achieved. The usefulness of the classical tool, the elastic theory, is exhausted and a dual obstacle blocks the way to further improvements. This obstacle consists of a lack of a clear definition of principles, and of a lack of comprehensive experience with the field of postelastic behaviour.

A discrepancy exists between the assignment of efforts to the various phases of ERCBC and to their relative importance. Great sophistication and emphasis is concentrated on the determination of a set of loads by means of algorythms based on the elastic theory. They deliver one particular model of dynamic behaviour which immediately after, is qualified as not being the right one, because one can not afford to make structures that stand up to its demands. (a notable exception in this respect are structures related to nuclear reactors which are designed for fully elastic and uncracked behaviour during a 1000 year earthquake, or similar criteria. Perhaps rightly so, the fear of the general mess created by a fractured nuclear vessel is so much greater than for the structural collapse of another building, that the additional cost in materials is acceptable).

Faithfully, designers have developped ways and means to analyze the elastic model, to a considerable degree of sophistication and capacity. Large computer programs exist capable of analyzing substantial structures dynamically with considerable accuracy. Interaction with a given earthquake record can be calculated without undue difficulty. However, this is where everything comes to a halt.

Beyond the elastic model and its possiblities only a maze of factors and extremely crude empirical procedures can be found to represent the inelastic range. Because no consistent concept of these events seems to exist, contradictions, inconsistencies and gaps are left in the coverage of design cases by these rules. For the same reason these gaps and inconsistencies cannot be resolved by the designer. Knowledge of the principles is needed, and only when the key terms will be explained and defined clearly can logic be employed in ERCBC. Without these principles no set of empirical rules will ever be consistent and complete but contradictory and complex.

ACKNOWLEDGMENTS

The writer whishes to express his appreciation for the help he received towards this paper from Robert Spokowski who assembled all the analytical data and reviewed the paper. Thanks to Monique Corby for typing the maniscript and to André Vézina for drawing the figures.

Thank to all of them.

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TABLE TO FIG. 1

	floors & bays	structure	height		sources
1.	3 storey, 1 bay	portal frame	35'		author's office
2.	10 storey, 1 bay	portal frame	120'		"
3.	12 storey	walls	105'		"
4.	11 storey	wall - high shear area	152'		"
5.	11 storey	wall - low shear area	152'		"
6.	40 storey	wall - frame	540'		"
7.	25 storey	frame	310'	)	
8.	15 storey	shear wall - frame	178'	)	* J.H.Rainer (3)
9.	14 storey	shear shall	155'	)	
10.	10 storey	wall + flat plate	100'		author's office
11.	26 storey + observation tower	wall + frame	460'		Tso & Bergman (1)
12.	3 storey	wall + flat slab	40'		author's office
13.	3 storey	flat slab	40'		"
14.	2 storey	flat slab	25'		"
15.	CN Tower	multicell shell	1800'		"

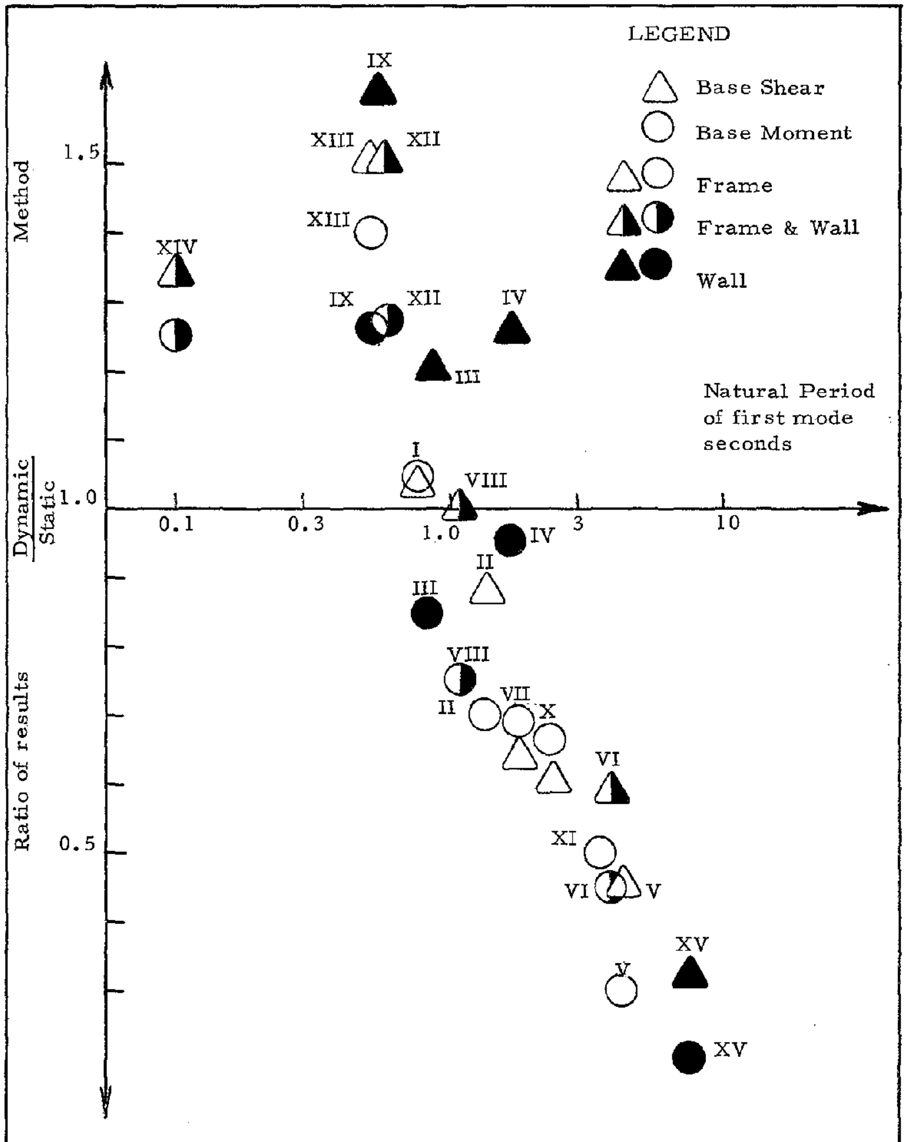

FIGURE I
Comparison of Results
Earthquake loads by different methods

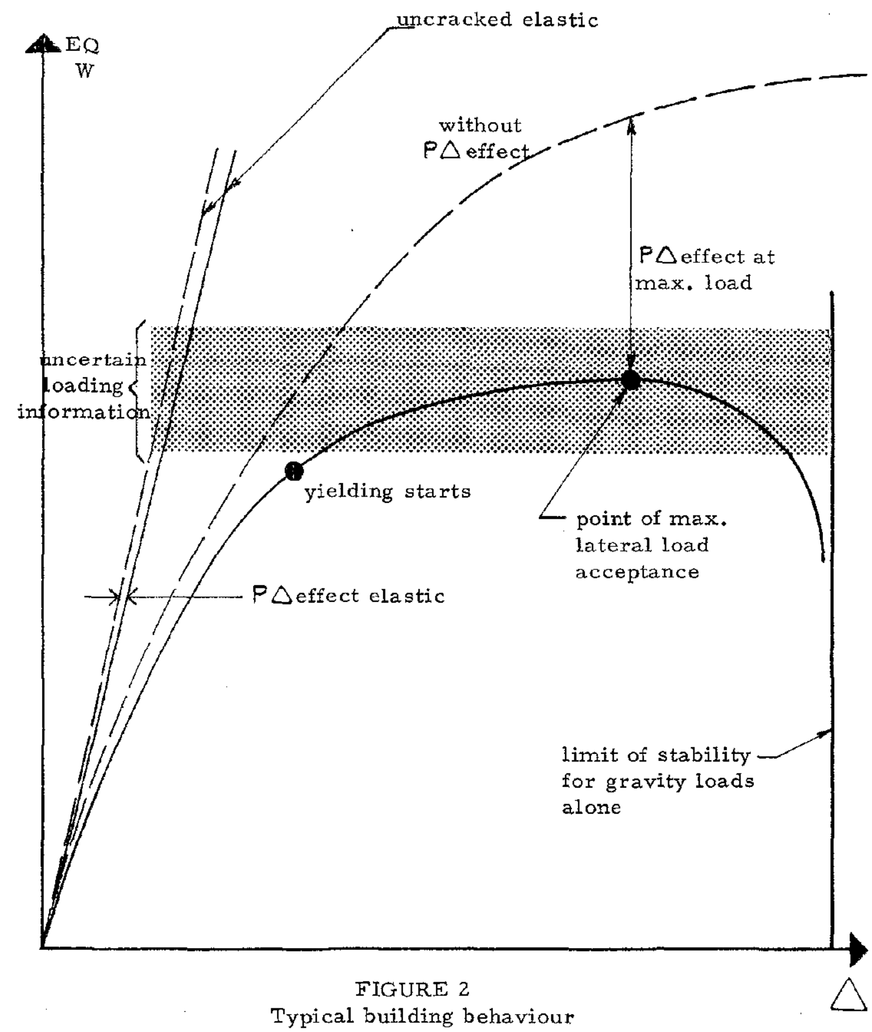

FIGURE 2
Typical building behaviour

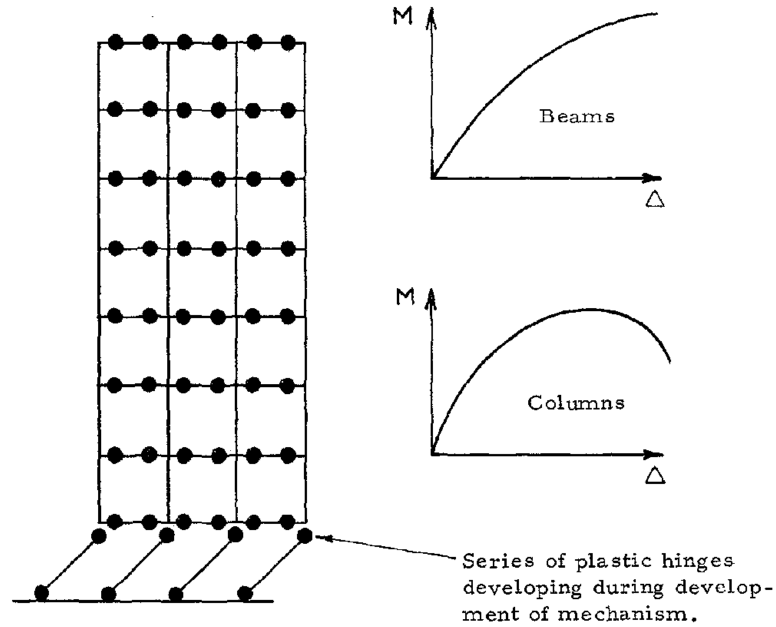
M
Beams
Δ
M
Columns
Δ
Series of plastic hinges developing during development of mechanism.

FIGURE 3

USER NEEDS FOR IMPROVING EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION

by

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INTRODUCTION

There are many areas of Earthquake-Resistant Reinforced Concrete Building Construction where our knowledge of the behavior of concrete components under the effects of earthquake motions is incomplete. I am going to confine my remarks on user needs to diaphragms and shear walls.

There has been a large concentration of effort on frame design, both in the testing of frames and frame components and in the development of computer analysis programs. The performance of shear wall construction in recent earthquakes has been relatively good in comparison with concrete frame construction. There have been areas where weakness or overstressing have been evident. Many of the areas of weakness do not provide ground for further research but only re-emphasize what has been observed many times in the past. Concrete construction must conform to the best practice and good control must supplement good design in order to accomplish this.

There are, however, adequate areas of uncertainty in the behavior of diaphragms and shear walls beyond those weaknesses to warrant considerable effort in testing and analytical development.

DIAPHRAGMS

Most analyses, including the majority of the computer programs in general use, are based on the assumption that the floor and roof diaphragms in reinforced concrete building construction are infinitely rigid. This assumption is probably valid for buildings in which the vertical seismic resisting system is in the form of moment resisting frames. There are other extremes where long slender diaphragms are used with low stiff shear walls where the assumption is clearly not valid. I believe that we need to establish criteria for determining the limits for which the assumption of infinite rigidity is valid.

In line with the need for establishing the above criteria we should review our procedures for determining the section properties of the diaphragms and the deflections which will result from applied loads. I believe the general practice is to consider the slab as the web of a deep girder and the edge beams or spandrels or walls as flanges. I have some reservations, considering the deep beam effects, on the validity of this procedure and would like to have the reservations resolved.

There is one publication, the Tri Services manual "Seismic Design For Buildings", which establishes a limit on diaphragm shears based on slab depth to span ratios. I am not aware of the background for the limitation and feel that investigation in this area is warranted. This investigation should establish whether there is need for such a limitation and the criteria for setting the limits if they are needed.

Most diaphragms have openings which should be considered in their design. These might be considered much the same way we account for openings in the webs of steel girders. The question again arises as to the applicability of these procedures with deep beams. I believe a program of testing of diaphragm models with various sizes and locations of openings simulating the more typical construction layouts would be beneficial. This testing would provide data on the effect of the openings on diaphragm deflections and on localized stresses at the openings. The data could then be used to formulate criteria for the design of the "chords" at the edges of openings, including the required extension of the "chords" beyond the edges of the openings.

Diaphragm deflections affect structural and non structural components connected to the diaphragm. Excessive deflections can adversely affect the performance of structural components and the connections of non structural components. The out of plane bending of a shear wall would be an example of a structural component that requires a limitation on diaphragm deflection to assure that the wall will retain its seismic resistant capacity for in plane forces. There is a need to estatablish the deflection limitations for the various components which will be attached to the diaphragms and at least the one I have mentioned is directly concerned with E R C B C.

SHEAR WALLS

Many of the areas of uncertainty in the behavior of shear walls are similar to those for diaphragms and can be answered by the same sets of tests or investigations. The effective "flange" areas to be combined with the shear wall "web" is one common area.

Many shear wall configurations have "flange" areas which are connected to other components by "coupling beams" and these have an effect on the actual stiffness of the shear wall as it responds to the earthquake motions. Experimental work with walls of this type could provide insights into their relative behavior in comparison to walls with flanges only.

There have been several occurances in past earthquakes where shear wall structures have exhibited evidence of rocking on their foundations. This type of response changes the displacements which the components of the structure experience. The distribution of seismic forces to the various components, where the components have different properties, determined with the rocking effects considered will differ from the distribution determined considering fixed base conditions. There are modeling techniques for simulating this condition using "soil springs where the soil properties have been established by site investigations. The modeling is based on theory and I am unaware of any testing which might verify the validity of the procedures. It would also be beneficial to have some simplified procedures for approximating these effects on smaller buildings when a full site investigation is not warranted.

Shear wall structures have, in the past, been considered to perform in a non ductile manner. There have been a number of papers on new techniques and reinforcement patterns which indicate that a high degree of energy absorption can be obtained without deterioration of the walls. There should be additional work done in the testing and evaluation of special shear walls.

Shear wall tests have been confined to components with or without end enlargements such as columns or flanges. The behavior of shear walls with intermediate columns and openings has not been established. I have been involved with a building having such a configuration. I would like to have test results from similar configurations to compare with my assumptions and provide guidance for others confronted with similar problems.

ANALYSIS

New computer programs which would account for the effects determined by testing of diaphragms and shear walls will be needed for maximum benefits to be realized from the experimental programs.

Present programs of which I am aware employ panel units in conjunction with frames or use finite element procedures. The latter procedure will provide a good model for a wall of constant thickness with openings. The "flange" effects and effects of "coupling beams" in the "flange" walls may not be adequately incorporated in such modeling. The results using models with infill panels in frames have not been completely consistant.

CONCLUSION

The workshop should provide a good forum to review the needs for research to improve Earthquake-Resistant Reinforced Concrete Construction. I hope that my questions may provide some grist for the mill.

There is one area where the workshop may be of major help to people like me. The volume of research which is conducted is not readily available and where it is available there is insufficient time to review the material. A compilation or synopsis of the research and findings would be welcomed by the practicing engineer.

USER NEEDS

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EARTHQUAKE RESEARCH AND USER NEEDS

by

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INTRODUCTION

In evaluating earthquake research as it pertains to user needs, both technical objectives of specific research projects and broad societal goals, such as reducing injury, life loss, and the socio-economic dislocations following a major earthquake, must be considered. As the task of reviewing technical objectives has been assigned to the workshops, my task is to evaluate earthquake research and its relationship to user needs on a broader scale.

Research programs dealing with earthquake resistant reinforced concrete building construction cannot be separated from other areas of research that bear on the overall seismic hazard of concrete buildings. Subjects related to assessing and reducing life hazard and damage, such as seismic zoning, earthquake risk analysis, hazard abatement in existing buildings, hazards in nonstructural components of building systems, and concomitant earthquake hazards such as fire and release of toxic chemicals, must be considered.

In assessing the relationship between earthquake research and user needs, a number of questions must be answered. Some important questions are:

1. Are researchers and users asking the right questions?
2. Are users taking advantage of research results?
3. Can the rate of achieving the societal goal of mitigating hazard be improved, and if so how?

Although the above list by no means exhausts the number of questions that must be answered in order to assess fully the degree of success of current research programs, they reflect primary concerns. Given the limits of this review and the subjective nature of answers to the above questions, the following assessment is not definitive.

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ARE RESEARCHERS AND USERS ASKING THE RIGHT QUESTIONS?

To clarify this question it is necessary to examine who are the researchers, who are the users, what questions have been asked, and what criteria would be appropriate to judge the aptness of such questions.

A researcher must have two essential characteristics: the ability to inquire carefully into a well-defined subject area, and to disseminate results of this inquiry through publication.

Research is not limited to schools and universities where faculties are engaged in (more or less) careful inquiry, trying to discover (more or less) relevant facts about a specific subject. An equal and perhaps even greater number of researchers are engaged in similar investigations in governmental, industrial, and private research institutions.

Two distinctions may be drawn between university and nonuniversity researchers. University research also serves as training for students who in the future will become either researchers or practicing professional engineers. The university researcher is more often than not a part-time researcher and part-time teacher or student and is under considerable pressure to publish. Publication is his bread and butter, just as meeting client needs is that of the design professional. The nonuniversity researcher is more likely to pursue short-term, mission-oriented inquiries. While he may be a full-time researcher and is often protected from the publish or perish syndrome, this protection has the effect of delaying publication.

Although it is more difficult to define users of research, two main categories may be identified. The first consists of researchers who must assimilate prior findings in order to maintain continuity in newly initiated inquiries. In the second category are those who seek to translate research findings into practice. Representatives of industry and industrial research organizations who participate in translating research findings into practice through serving on various technical committees fall in this category. Professional engineers who keep abreast of research findings in areas directly or indirectly related to their professional practice and are often not reimbursed for their time also fall in this category. While representatives of academic research groups also serve on technical committees, it is more often than not the representatives of industry and practicing professionals who are chiefly responsible for the content of codes and standards that affect the safety and cost of structures.

Another user group that may not participate actively in translating research into practice, but is nevertheless important, consists of designers responsible for all phases of planning, design, and construction of projects, and consultants on special aspects of design projects. Here again the user and researcher may be the same individual, wearing different hats depending on the task being performed. Finally, the most important although passive user group is the public, including private investors, public institutions, and occupants.

It is essential to determine whether researchers and users are asking the right questions, i.e. whether solutions to their questions help to achieve societal goals of mitigating life hazard and reducing socio-economic disruption. Questions asked by users are more difficult to identify than those asked by researchers. It may be assumed, however, that users' questions are communicated to researchers in formal and informal ways, and are reflected in work carried out by researchers. In Table 1 (page 6), a list of subject areas related to earthquake engineering research and reflecting questions asked by researchers has been compiled partly from the agenda of this workshop and partly from general subject areas covered in earthquake engineering publications.

Whether researchers are asking the right questions is determined by the extent to which appropriate solutions to salient problems are provided. It is relatively easy to demonstrate that practice has improved remarkably as a direct result of research conducted over the last fifteen to twenty years.

1. Studies of seismicity, strong motion records, and earthquake risk have substantially improved seismic zoning maps.

2. Advances in geotechnical and structural engineering have enabled designers to account more reliably for soil-structure interaction.

3. The importance of higher modes of dynamic response in some structures has been generally recognized and, where appropriate, can be accounted for in design.

4. Greater knowledge of energy absorption capacity and failure mechanisms of different structural systems (frames, shear walls, and combined systems) has led to improvements in design criteria, material selection, and detailing requirements.

5. Field observations, including vibration measurements on buildings and comparisons of observed damage with predicted response, have also contributed greatly to improvements in design criteria and detailing requirements.

While this list includes only a few instances of significant advances and improvements in design directly linked to research results, it can be concluded that researchers are providing at least some appropriate solutions to salient problems, and thus have been addressing meaningful questions.

There are, however, significant gaps in current research, and some areas already under investigation must receive greater emphasis. From the user's point of view (where the user is defined as the public at large and not just as the design profession), three main areas must be addressed:

1. Damageability as a limiting design criterion for both structural and nonstructural building elements should be defined quantitatively.

2. An integrated hazard evaluation wherein earthquake-related events (failure of mechanical systems, release of toxic materials, fire) are accounted for should be developed. The risk of seismic hazard cannot be properly assessed and hazards can only be partially reduced until concomitant risks are considered.

3. A better method of assessing potential earthquake response of structural systems in existing buildings that do not conform to 'ideal' systems being researched should be devised. Some of these building systems are no longer permitted under codes, but are frequently encountered in existing buildings. To estimate the risk represented by such buildings, more precise information on performance is required.

Research programs related to the first two areas have been initiated, but the writer knows of no broadly based research program dealing with the third. Given the past performance of the research community, these challenges will not go unheeded long, and solutions will be forthcoming.

ARE USERS TAKING ADVANTAGE OF RESEARCH RESULTS?

Design criteria for earthquake resistant structures, particularly for reinforced concrete structures, have been improved significantly. In successive editions of SEAOC "Recommended Lateral Force Requirements and Commentary," in proposed revisions of Appendix A, "Special Provisions for Seismic Design" in the ACI Building Code Requirements for Reinforced Concrete, and in ATC's "Final Review Draft of Recommended Comprehensive Seismic Design Provisions for Buildings," it is clear that research results have been employed extensively by users. However, research results are applied neither universally nor rapidly in practice. Delays in applying research findings arise from the need to evaluate and interpret results critically before they are accepted into practice. A healthy approach to innovation must proceed with deliberate speed, and must be accompanied by a careful evaluation of real benefits; innovation without improvement has no merit.

A case in point is that of the development and gradual acceptance of computer programs for analyzing the dynamic response of structures to earthquake motions. A great many designers have used such programs to evaluate and refine preliminary designs. While these methods have not and may never be accepted universally, the number of structural engineers using such programs has been growing steadily.

Results of some investigations may be useful in advancing knowledge of parameters affecting structural response without finding specific application to design problems. The importance of such research is in no way diminished.

Users must bear a good deal of the responsibility for critically weighing and evaluating research, and for finding appropriate applications. In a few areas, users have not yet recognized the potential of research results to improve practice. The interdependence of structural response history and structural characteristics is not sufficiently recognized. Most designers calculate loading or response to loading without considering changes in structural characteristics that are dependent on loading history. Thus, effects of cracking or changes in stiffness are ignored, as are effects of repeated cyclic inelastic deformation (low-cycle fatigue) on strength and energy absorption capacity. Both analytical and experimental research clearly demonstrate the interdependence of structural resistance and loading history, yet they are treated separately in most codes.

The need to integrate seismic hazard evaluation with assessment of concomitant hazards, such as fire, has also been largely ignored. Traditionally, fire and earthquake resistance have been considered separately, leading to a number of design anomalies and an overall reduction in safety. If it were recognized that fire and earthquakes are closely related, refinements in criteria that would simultaneously simplify design and increase safety could be readily introduced. For example, a requirement that one-third of maximum positive reinforcement and maximum negative reinforcement be continued beyond theoretical cut-off points would greatly reduce potential damage and hazard of collapse due to seismic load reversal and thermal gradients from fire. Other recommendations are more difficult to formulate, although available analytical methods could be used to develop such provisions.

CONCLUSIONS

In considering how the rate of achieving societal goals of hazard mitigation might be improved, I find no reason to conclude that this rate has been too slow. On the contrary, given the magnitude of the task, remarkable progress has been made in translating research results into practice. Nevertheless, certain measures might improve the current rate of assimilation:

1. Research efforts with long-range objectives must be increased to support any program attempting to implement results more quickly.

2. An increased rate of implementation will require greater numbers of professionals with the capability, training, and experience to evaluate and screen research.

3. Closer cooperation and more effective communication between researchers and users are essential. Research advisory committees and professional consultants help to promote communication, but such input is necessarily limited by the short period of engagement. Designers must be willing to accept leave from practice and to engage in full-time research for periods of one or two years. Conversely, researchers taking leave from universities must be willing to engage in full-time professional practice for similar periods of time.

TABLE 1 SUBJECTS OF EARTHQUAKE-RELATED RESEARCH PROGRAMS

I. Engineering - Seismology

- Seismicity, Strong Motion Records
- Seismic Regionalization and Zoning
- Local Site Seismic Characteristics
- Earthquake Risk (statistics, probability analyses)

II. Dynamics of Foundations

- Soils and Rocks - dynamic properties and behavior
- Soil-Structure Interaction
- Foundations - Footings, Rafts, Piles, Retaining Walls - dynamic behavior, earthquake response

III. Dynamics of Structural Materials

- Mechanical Properties and Performance
- Concrete
- Steel - reinforcing and prestressing

IV. Dynamics of Structures

- Conceptual Choice of System - foundations, moment-resisting frames, floor systems, frame-wall systems, prestressing, prefabrication
- Methods of Analysis - preliminary design, linear and nonlinear dynamic analysis, nondeterministic analysis, modeling and computer programs

V. Experimental Investigations - Comparison to Analytical Predictions

- Reduced Scale Subsystems - foundations, moment-resisting frames, floor systems, frame-wall systems, prestressing, prefabrications, elements, and subassemblages
- Large or Full-Scale Subsystems - subassemblages and model buildings

VI. Field Observations

- Vibration Measurements
- Effects of Earthquakes on Structures - Damage
- Comparison of Observed Damage to Predicted Response

VII. Design and Construction

- Codes and Standards
- Quality Assurance

VIII. Natural Disaster Hazard Mitigation

- Damage Assessment and Repair
- Hazard Abatement in Existing Buildings - Strengthening
- Integrated Hazard Evaluation - earthquake, fire, tsunami
- Earthquake Prediction
- Socio-Economic, Legal, and Political Aspects

APPLICABILITY OF EARTHQUAKE RESEARCH FROM THE USER'S VIEWPOINT

by

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INTRODUCTION

The purpose of this paper is to review research that has been performed related to the earthquake resistance of reinforced concrete structures and comment on its applicability from a user's viewpoint. If this paper were to review and comment in detail on the many recent projects in this area, it would take months to prepare, fill a book, shatter many egos and probably not contribute anything constructive to this workshop. Therefore, this paper will present some general comments on the applicability of completed and present research, on what the user needs and expects from research in this field and how, perhaps, future results can be made more usable.

GOAL OF EARTHQUAKE RELATED RESEARCH

The primary goal of earthquake related research is to improve life safety considerations in the event of an earthquake. This goal is simply to save more lives. A secondary goal is reducing the damage in structures subjected to earthquakes, thereby reducing the resulting economic loss. One might say that the goal of a particular research project is to learn about the performance or strength or characteristics of a particular structural element, system or material. However, the ultimate goal is to utilize that research in providing safer buildings for the public in seismic areas. It is this ultimate goal on which I will evaluate the appicability of research.

There are two primary types of research - basic and applied. Basic research is the study of problems simply to learn about them, without any specific application. Although we depend on much basic research in earthquake engineering, I am not going to discuss it in this paper. Rather, I will concentrate on applied research - the transformation of basic research with additional experiments and studies into results which can be applied to real, practical problems. This is the research of use to the structural engineer to fulfill the basic goal of providing safer buildings.

The needs of the profession and the potential users are so extremely important in planning and executing a major research project that it really shouldn't need mention. Although, too often these considerations are overlooked. The user, the practicing profession, is looking for detailed studies with clear, concise conclusions presented together with all limitations of the research. He is looking for simplier ways to analyze or design, not more complicated methods. He is generally not interested in complex analysis which comprises pages of partial differential equations in the ASCE's Structural Journal. He would prefer that "publish or perish" in academic circles was not a reality. He is looking for practically oriented results which provide safer buildings, which reduce design efforts and which do not omit important limitations which might cause a project to have problems and resulting litigation. These needs and desires of the profession must be understood if research is to be useful to the practicing professional.

THE APPLICABILITY OF RESEARCH

Considerable earthquake related research is currently underway and much more is anticipated, especially with the possibility of increased federal funding in this area. Some of the recent research has been excellent and is finding its way into practice. However, an equal or greater amount often misses the need and becomes little more than another volume to line a library shelf. This paper hopes to touch on a few points which might improve the usability of such future volumes.

Possibly the most important aspect of generating useful earthquake related research is properly conceiving sound research in the beginning. If the researchers does not start off with a sound and reasonable approach, things will only get worse. On occasion I receive research proposals from the National Science Foundation for review. I am often amazed at the complete lack of understanding of a problem that is displayed in some of these proposals. Some researchers tend to have no idea how structures are designed nor how they perform. This Ivory Tower image is very real and, unfortunately, results in some low quality research of limited practical value.

One way to improve this situation is for the researcher to gain some practical experience and knowledge. Some of the best research generated is by researchers who have spent a few years during their careers designing structures and working in a practically oriented design job. How many research professors have considered spending their sabbatical year working in a consulting engineering office performing routine and/or specialized structural engineering chores? I don't mean performing sophisticated computer analysis of some special or unusual problem, but actual building design and detailing and possibly repairing a few distressed structures. I realize that this is not as glamorous as spending a year in some foreign country as a visiting professor or lecturer and doesn't gain the international awareness, but it might result in more practically oriented research which would certainly benefit the profession.

The Advisory Committee is another method which can be of some benefit in steering research along a practical road. Such committees are becoming more widely used, especially on NSF funded projects. The Advisory Committee can help, but it must be able to supply strong input at the beginning of the project. The researcher or research team must also be willing to accept some advise and consider alterations to the research plan, which is not always the case. Furthermore, the Advisory Committee usually does not receive sufficient progress reports or information to keep them up to date on the progress of the program. The receipt of volumes of reports one week before a once a year meeting is hardly adequate, but it is the way it is often done. For Advisory Committees to be effective, they must be willing, kept well informed, and be able to offer advice to a researcher who is grateful and will consider and use advice.

Another variation on the Advisory Committee which might be helpful would be having one to several practicing engineers act as consultants on a research project. They could spend two to four days full time with the research team several times a year to offer advice and comment. Useful brainstorming and data manipulation sessions could prove fruitful. Obviously, these consultants would have to be reimbursed their usual consulting fees, but such an investment might be well worthwhile. Obviously, the key time for such input would be at the beginning of the project.

Improved coordination and review between researchers may also be in order. I know that researchers converse about common problems and share results as research is in progress. However, if this effort were expanded where researchers would take turns visiting other universities or laboratories for three to five days or so and thoroughly review and critique the work being done. A fresh mind often sees things in a new way, and fresh light can be shed.

Another approach which needs more consideration is the multidisciplinary team for complex research projects. In this day of specialization, a single person is unable to tackle all the problems. Many of our research needs involve topics where seismologists or geophysicists or geotechnical engineers or material specialists or some other specialist must team up with the structural research engineer to understand the problem and work as a team. If we are to tackle many of the important research needs over the next few years, multidisciplinary research teams must be formed or fruitation of usable results will never result. Again,the practical understanding of the problem is essential for all members of such teams, as the theoretically oriented researcher will not be able to contribute fully to solving the practical problems.

A brief word also seems to be in order on research funding, as I feel that it may have an effect on the usefulness of research. In past time, research was funded largely as a part of a university's basic program. Outside funding of major projects was often in the form of donated reinforcing steel, structural steel, or bags of cement. The salaries of all concerned came from the institution's budget. Lately, with the pressure to reduce state and local tax expenditures, there seems to be a great demand for federal or industry grants to underwrite the entire research effort. The quality or usefullness of the research sometimes seems secondary, the primary goal being simply to obtain funding on enough projects to pay salaries, overhead, and perpetuate the department. It is realized that research institutions are big business and continued funding is essential, just as a continual flow of commissions is essential to a design office. However, this reality of current life does not always foster research studies useful to the practicing design profession. The publish or perish urge is a similar situation.

FROM RESEARCH TO PRACTICE

Once any research project is completed and the report is printed, there is still a long trail to follow before the research is applied and safer or more economical structures result. The first step is disseminating the information to those who are interested. This is usually accomplished by articles in technical journals and limited distribution of the original reports. With the one year plus delay process for publishing in most journals, speedy dissemination is hardly possible. The direct distribution of the original reports to knowledgeable engineers and researchers is essential and such efforts should be expanded. Research funding should perhaps include increased amounts for such disseminations of useful material.

Some research results may be in areas where building code changes must be considered based on the new information. Such code changes might be either to relax certain provisionsor make others more restrictive. In either case, considerable study is in order by code writing and other technical committees to evaluate the full impact of the new information. Quick code changes are often regretable code changes. In the areas of non-seismic design, considerably more research is available with which to compare the new information. However, in the field of seismic design, research is relatively limited so one has to approach code changes with greater care and more study. The limitations of the research must be fully considered before any codification.

Writing reports and proposing code changes is not the whole answer. However, there is a definte need to disseminate new ideas and techniques and useful research results to the practitioner who has not been reached by the previously mentioned methods. The need for low cost, practically oriented local seminars is present and must be fulfilled. University extension courses often charge too high a fee for the average engineer and industry groups or professional organizations often fill this need. It is essential that the useful information reach the individuals who can apply it to newer and safer designs.

CONCLUSIONS

In conclusion, it is the opinion of the author that earthquake engineering research can often be more applicable for usage by the profession. Methods which might improve the applicability of such research are:

1. Properly conceiving useful research to fulfill known needs.

2. Researchers should improve their knowledge of practical structural engineering design.

3. Advisory Committees can be of assistance if the committee is kept well informed and the researcher is willing to consider their advice.

4. Practicing professionals acting as consultants to a research project with detailed input may have advantages, but funding must provide for compensation.

5. Improved coordination among researchers including critiques of each others studies.

6. Multidisciplinary research teams are essential to understand and solve many complex needs in earthquake engineering.

KEYNOTE ADDRESS:

SOCIAL AND ECONOMIC EFFECTS OF EARTHQUAKE PREDICTION

by

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ABSTRACT

In order to plan for the release and implementation of earthquake warnings we must be able to forecast both individual and organizational response to the announcement, and then the response to the response. This is further complicated because many individual and organizational actions are responses to anticipated rather than actual responses.

In order to understand public response to earthquake predictions and warnings we must first appreciate what people are likely to hear, how they will understand it, and what they will retain. People typically mix physical or scientific and nonscientific frameworks in understanding earthquakes. First reactions to an earthquake announcement include assimilation of the announcement to prior personal experience and seeking to confirm the danger through the testimony of one's own senses. Fear and concern evoked by announcements are complex and often contradictory. Preliminary evidence from a study of Southern California response to announcements about the Mojave Uplift suggests that earthquake concern has low salience in people's lives but evokes considerable fear when the topic surfaces. Panic in the sense of sustained disorganized flight behavior on a mass scale is an extremely rare phenomenon, and is unlikely to follow earthquake warnings. Inaction rather than panicky overreaction is more likely to prevail, because of the lack of realistic alternatives to customary behavior, because of the prevalence of "satisfying" rather than "maximizing" responses, and various other reasons. Disaster threats of many kinds provoke a Denial-Inaction syndrome of normalization and seeking the familiar. In a reciprocal relationship inaction fosters disbelief and disbelief impedes action. Informed and collaborative public response is also impaired by the gap between an elite scientific community and the public, especially in a society which is otherwise committed to democratic values.

Governmental and business organizations are likely to resist the issuance of warnings when faced with probable but uncertain disaster. Organizations charged with emergency response and hazard reduction typically respond more with continuity than with innovation in crisis situations. Research by sociologists Haas and Mileti suggests that and earthquake prediction issued with reasonable confidence and an extended lead time may instigate a substantial economic recession in the affected area. In a real situation the corporate behavior leading to these consequences may be modified by both unplanned competition for long-term markets and government intervention and support.

Cooperative popular response to government-coordinated hazard reduction programs was strong in China, probably in part because of popular involvement in the prediction enterprise--something that has not occurred on a large scale

yet in the United States. Scapegoating against public officials can occur when there is prior polarization on other issues, but is less likely than often supposed. On the other hand, there is no tradition for viewing earthquake threat as a condition requiring collective action rather than individual and family adaptation, and there may be insufficient basis for emergence of the altruistic sentiments common after disaster. It may be easier to develop grass-roots organizational collaboration to opposed government-instigated hazard-reduction steps than to support them, as illustrated already in Southern California.

There is need for a differentiated approach to earthquake warning and hazard reduction, noting that danger, need, and ability to cope are quite unequally distributed throughout the population. There is a danger that we will do most for the many who can cope effectively at little personal expense while overlooking sizable minorities for whom the problem is unmanageable.

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MECHANICAL CHARACTERISTICS AND PERFORMANCE OF REINFORCED AND PRESTRESSED CONCRETE MATERIALS UNDER SEISMIC CONDITIONS

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

MECHANICAL PROPERTIES OF CONCRETE

by

F. Robert Preece

Executive Vice President

Testing Engineers, Incorporated

INTRODUCTION

The last words of Buddha were "All composite things decay - Strive dilliently".

Concrete is a composite of composites - its mechanical properties, withut reinforcement, are time dependent as well as dependent upon environment, umidity, temperature and winds. When reinforced with steel its static mehanical properties alone change in sometimes surprising and varied manner. hen subjected to earthquake motion concrete structures undergo random, chaoic and erratic vibratory motion that is dependent upon magnitude and type of nergy release, epicentral distance, type of fault slippage, depth of focus, hysical properties of underlying foundation soils; and geometrical configuraion, mass distribution, and quality of workmanship of the building. What a ertile field for research! We must indeed strive dilligently!

BASIC INGREDIENT MATERIALS

In the discussion that follows, it is presumed that everyone is thorughly knowledgeable of the standard specifications and routine tests for uality of the basic material components: Cement, aggregates, water, admixures as well as those tests performed on concrete freshly made, and hardend concrete tested at specified ages. The present state of the practice can e summarized by saying that all of the published information is readily availble to produce strong, durable concrete of uniform quality. The efforts of ich organizations as the American Concrete Institute [1], [2], the Associaion for Testing and Materials [3], [4] and the Portland Cement Association [5] re recognized world wide. I cannot praise them enough. In spite of these eadily available standards, much misunderstanding and myth still prevail howver, among designers as well as workmen and their supervisors as to good workmanship practices and the definition of quality concrete.

UNCONFINED COMPRESSION

Current practice, because of code provisions, can be divided into two cateories - inspected concrete, and uninspected concrete. For uninspected concrete t is possible for a designer to specify 28 day design strength (f'_c) of 2000 psi, ounting on the contractor to produce concrete (for reasons of workability alone) n excess of 3000 psi. On the other hand for inspected work a more prudent designer may specify a 4000 psi strength for all concrete assuming some tests will e as low as 3750 psi, upon which his design is actually based. Normal practice ies somewhere between those two extremes. There seems to be a general mistrust mong designers regarding the current ACI statistical methods for evaluation and acceptance of concrete quality. I appreciate the desire of designers to view concrete strength as a single number, all the design aids are devised on that basis. The designer, faced with the complexity of decision making, must try to simplify his task by reducing material properties and earthquakes to single values in order to complete his job on time.

The simple unconfined compression test of concrete, cast in a cylindrical metal mold, six-inches in diameter and twelve-inches long, tested at 28 days in the laboratory after curing at 70°F and 100% relative humidity, is the standard index for quality of concrete delivered to the jobsite. The actual concrete quality or strength in the structure may vary widely from that index. There are some continued efforts to avoid the "lunacy" of waiting 28 days for acceptance by accelerating the strength gain. This is done by drastically changing the environment of the standard test specimen. The aim is to obtain acceptance of the delivered concrete within one or two days after placing it. This commendable approach has been met with non-support and some resistance by the designers who maintain they can't possibly accept the concrete until it is properly cured, in spite of the fact that the specification requirements for curing are those most often ignored.

In San Francisco it is possible to produce workable normal weight ready mix concrete of 3000 psi class consistently averaging 3600 psi, having a standard deviation of about 250 psi (7%) using cement contents of 4.3/4 sacks per yard with common water reducing admixtures. It was not always possible. Our several unpublished studies of existing concrete buildings constructed 30 to 50 years ago reveal concrete strengths with a range from 1500 psi to 4000 psi in the same building. Coefficients of variation on such concrete often exceed 30%.

We have encountered some difficulties interpreting ASTM C42 [3] which governs the sampling and testing of hardened concrete. It is silent regarding the adjustment of test values with respect to age, size, shape of specimen and effects of coring operations. Corrections for length-to-diameter ratio are given only for the range from 1.0 to 2.0. As a consequence, when specimens are taken from relatively thin sections - slabs on metal deck for instance, no guidance is given for evaluating the concrete in place. Some investigators [6] soak specimens in lime-saturated water for 1 to 3 weeks then dry-out the specimens for a similar period to heal small cracking induced by coring.

We have no test or criterion for evaluating relative "toughness" of plain concrete in the field. The ability of some concretes to absorb energy of deformation without bursting has been observed by the writer. We have noted the manner of failure of high strength, low modulus, lightweight concrete - strengths of 5000, 6000 or 7000 psi are obtainable in one-day stream-curing operations. When tested in unconfined compression this concrete fails rapidly by bursting dramatically in the machine, before the load can be removed. Bressler & Bertero [7] point out that confinement of lightweight aggregate concrete may produce increases in compressive strength of less than half those of normal weight concrete. The unconfined compression strength test therefore, is not a suitable predictor of performance of concrete to resist earthquake loading.

SPLITTING TENSILE TEST

This relatively simple inexpensive test may be a more suitable indicator of earthquake performance than any other available standard test procedure. Related as it is to shear and bond capacity through failures in diagonal tension, it provides an understandable model of the typical failures of brittle materials observed universally in all earthquakes. The test is particularly sensitive to types of aggregates as well as aggregate to cement bond capacity. Although not originally intended as a field acceptance test, its use in evaluating lightweight aggregates for design purposes demonstrated that it could easily be used for job control purposes. Study should be made for its extended use as such.

FLEXURE

Because of sensitivity to shrinkage stresses, discontinuities caused by aggregate segregation and awkward size and weight of specimen, the flexural beam specimen commonly used to test pavement concrete does not appear to be particularly suitable for evaluating building concrete. Wide variations in reported strength is common with this test procedure. The splitting tension test would appear to be a suitable replacement as an <u>index</u> of strength for paving.

MODULUS - OF ELASTICITY IN COMPRESSION

This is not a routine test in current practice. Some designers have specified Young's Modulus for lightweight concrete of 2.5 X 10^6 psi minimum, without specifing age, test procedure or method. They were astonished to find out that at 28 days for 3000 psi concrete using Secant Method at .45 f_c' the reported test modulus was as low as 1.75 X 10^6 psi. The only organized effort to evaluate modulus of elasticity of field produced concrete was for BARTD*elevated structures. The Standard Specifications for that project required that the Modulus of Elasticity at loaded age for the concrete proposed for use could not be less than 95% of a BARTD Standard Mix when tested after 28 days of drying. This requirement proved to be unreasonably restrictive, partly because the standard mix was not air entrained, yet project specifications for field concrete required air entrainment. This experience pointed out that with comparable unit weights and strengths, wide variations in Modulus of Elasticity (± 20%) could be encountered. Bertero and Bressler also report wide variations in concrete sampled from Olive View Hospital [6].

This test is relatively expensive to conduct, about three to four times the cost of a concrete compression test, and greater care is required by the laboratory technician in performing the test. Because designers may need to know the values with greater degree of certainty for predicting structure performance during quakes, it is possible that this test may come into more common usage.

DRYING SHRINKAGE AND TEMPERATURE

There are strong indications that tensile strain limitations determine the strength of concrete. This limiting tensile strain is usually assumed to be between 0.01% to 0.02%. The point of initial cracking is about the same magnitude for the tension face of a beam in flexure and the circumferential

*BARTD - Bay Area Rapid Transit District face of cylinder in uniaxial compression [8]. Poisson's ratio varies between about 1/8 for higher strength concrete to 1/5 for lower strength mixes.

With reported unrestrained drying shrinkage strains of from 0.04% to 0.08% [9] it is easy to understand that concrete in real structure becomes a linkage of discrete elements, connected by reinforcing steel across cracks as a result of drying shrinkage and thermal cracking.

Large [10] points out that normal creep considerations alone on concrete columns can easily double the stress in vertical reinforcement. This writer has observed transverse tensile cracking in concrete columns exposed to "normal" Bay Area drying conditions. He has also observed the tensile splitting and spalling of cover that occurred in a first floor column of a five-story building under dead load only. This occurred because misplacement of lateral ties permitted the buckling outward of a #18 corner rebar because the concrete shrunk, causing all of the dead load to be carried by the reinforcing steel. The nominal concrete cover alone could not keep the bars from buckling.

Most California buildings exist in less than desirable environment with drying winds and widely varying temperatures. This results in a continual movement opening and then partly closing construction joints and cracks. The resulting cracked structure bears no resemblance to the idealized elastic model as tested in the research laboratories or analyzed by simplistic computer methodologies.

Drying shrinkage testing of small prisms has become routine in San Francisco Bay Area laboratories, because of the effort of the Structural Engineers Association to place some control on drying shrinkage by restrictions on this measurable property [9]. However there is some disagreement as to the application of the reported values to real structures. The test is not particularly reproducible because of: 1) The stochastic nature of concrete. 2) The variation in material components particularly cement with time. 3) The variations between laboratory humidity control rooms. 4) The variations within each laboratory control room. The results can vary by as much as 100% for identical mix designs performed using the same brands of materials produced at different times, tested in the same laboratory. This program of intensive testing has had a salutary effect on the local industry, however improvement in reduced cracking has been observed.

It appears that it would be useful to perform dynamic and cyclic testing of structures that have been allowed to age and crack at sections where drying shrinkage would normally occur prior to earthquake loading.

The thermal expansion and contraction coefficient of actual concrete mixes is not specifically known and it may well be a time dependent variable also.

A new standard, ASTM C827 T, has been developed by Committee C9 to test for early volume change of cementitious materials. It is suitable to measure the plastic shrinkage under any specified environment. The length change that can take place in 3 to 4 hours setting time may be as much as 20 to 30 times the drying shrinkage length change for 28 days of drying.

Polivka's [11] restrained bar cracking test is also an important tool in evaluating cracking propensity of concrete due to shrinkage. It has had far less use than it merits.

CREEP IN COMPRESSION

Creep, defined as the increase in strain under a sustained load, can be several times as large as the instantaneous strain at loading and consequently has an important influence on the long time performance of buildings. The amount of creep is directly related to the compressive stress and is highly dependent upon the age at loading, duration of loading, type of aggregate, environmental conditions and history of previous loading. Because of the variables involved, a standard test procedure is not in routine use for building construction. Some testing for specific bridge projects has been performed to enable deflection predictions to be made with greater confidence.

Concrete proposed for use on BARTD elevated structures was required to exhibit no more creep 28 days after end of curing period than 110% of a BARTD standard mix tested in the same manner. ASTM C512 was modified using three 6-inch diameter, 16-inch long test specimens, stacked three high in a creep frame loaded at 1200 psi tested at 50% RH, 73°F. For the most part, the mixes and curing methods chosen for the project had little difficulty in meeting the requirements. Much information regarding creep characteristics of locally available material was gained, however very little of this information was disseminated to the design profession. The creep index was for short term loading only (28 days after cure) and the average net creep under the conditions of the test at that age approached 0.03%.

Creep has the beneficial effect of stress relaxation under sustained strains. However the actual stress state after many years of sustained loading in a drying environment is not known. But, the design profession needs to know more about this admittedly complex problem.

CONCLUSIONS

Concrete composites can be likened to living tissue, at birth it is weak and carries little load and must be handled with care and feeding (water) so it can mature. Later it shrinks and under sustained loads its joints creak and the body sags and stoops. While not bio-degradable, it most surely continues to degrade until the earthquake strikes. By then the structure is particularly vulnerable to large order deformations, which can cause collapse.

Paraphrasing Buddha - concrete, being a composite thing, decays - we must strive dilligently so that we can understand why, and learn how to prolong its life.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED
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CONSTITUTIVE RELATIONS FOR CONCRETES UNDER SEISMIC CONDITIONS

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SOME PRELIMINARY REMARKS

Seismic disturbances of a structure may create special conditions which are not normally considered in design. These include the following:

(1) A rate of loading which is much faster than normal.

(2) A stress or deformation level much higher than normal.

(3) Cyclic loadings to high levels of load or deformation; the cycling being of higher duress but lower number of cycles (and often of lower frequency) than those encountered in the more usual "fatigue" studies.

(4) These cycles of loading may be of one sense or they may reverse.

(5) Combinations of stress which are not normally contemplated in design.

(6) A variety of loading paths in multiaxial space.

The behavior of concrete in the domain defined by these conditions (or any combination of them) has not been studied until very recently and such studies are few in number and limited in scope. Until such time as more extensive test data become available (and have been organized into a form suitable for incorporation into analysis procedures) the profession will probably have to adopt the following approximation procedure:

(a) Study the data from experiments which, although not exactly of the kind desired, nontheless may shed some light on the behavior of concrete under seismic conditions (for example, tests under moderate strain rates may indicate what to expect under higher strain rates). This approach, however, is fraught with the possibilities of misleading extrapolations--some examples of which are described below.

(b) Study all aspects of concrete behavior and attempt to understand the basic mechanisms of the response of concrete to loading. Such understanding may well be essential for a proper utilization of the results of step (c).

(c) When the appropriate experimental tests are performed the resulting information will be voluminous and complex. The cyclic multiaxial behavior of a nonlinear, strain-softening, discrete, cracking material such as concrete will be too complex for designers to hope that every possible combination of special conditions mentioned above can be studied experimentally. It is much more likely that data will be found for extremes of behavior and analysts and designers will need to devise methods for interpolating and extrapolating the data to other circumstances.

(d) Using the knowledge acquired in steps (b) and (c) produce an analytical model capable of satisfactory predictions for all conditions considered in design.

In addition to the special seismic conditions mentioned previously, a modern concrete analyst is faced with other problems including:

(1) The increasing use of lighweight concrete (LWC), ultra-high strength concrete (UHC), and other new materials. Most current analytical procedures are based upon research which was conducted in 1950's or 1960's. Since that time normal weight concretes (NWC) have been increasing in strength while other materials such as LWC and UHC are becoming more widely used.

(2) Much of the past experimental information is suspect for other reasons (for example the stress conditions were probably not what the experimenter believed them to be).

(3) The subtle but disturbing problem as to how to use the properties of plain concrete, which is rarely found in the field, when predicting the properties of reinforced or prestressed concrete.

In view of (3) it will become apparent that one cannot entirely separate the study of plain concrete from that of reinforced concrete. Thus, although the topic of this paper is plain concrete it will be necessary, in what follows, to refer to reinforced concrete from time to time.

RANGE OF CONDITIONS TO BE EXPECTED UNDER SEISMIC LOADING

A review of the literature--even if confined to the North American continent plus England; and to the past ten years--reveals that a substantial amount of information has been gathered concerning many aspects of concrete behavior and modelling but very little for the special conditions of seismic loading. It is of interest to estimate the range of those conditions which are appropriate to earthquake studies.

Strain (or deformation) Rate

The frequency content of earthquakes shows that the majority of earthquakes impose motions at rates between 1 and 10 cycles per second. Although the higher frequency shocks often occur earlier in the event, and are usually accompanied by accelerations less than the maximum, it seems reasonable to use 10 cycles per second as an estimation of the highest loading rate to be considered in design. For a 4500 psi (31 MPa) concrete with failure at 3000 microstrain* this corresponds, approximately, to a loading rate of 45,000 psi/sec (310 MPa/sec) or 30,000 microstrain/sec.

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*1 microstrain = $1 \times 10^{-6} = 1\ \mu\varepsilon$

For situations where slower rates of loading are deemed more critical previous studies have provided substantial information (see Table 1) at least for concretes in present (1977) use.

Load Levels

Since the primary concern of the engineer is safety, it is necessary to obtain constitutive information all the way to failure. Furthermore, since "failure" is a matter of definition it may also be claimed that tests should be conducted "beyond failure" or to "repeated failure" since the post-earthquake serviceability and safety of structures are almost as important as their ability to survive the initial seismic shock. The protection of people during aftershocks (when structural integrity is probably reduced and emergency services probably much less than ideal) and the economic impact of reserviceability versus demolition and replacement are self evident.

Cyclic Loading Under Multiaxial Conditions

It is clear that very few structural members will be subjected to purely uniaxial stress, expecially during an earthquake. However, the infinity of possible combinations of loadings means that it is impossible to investigate them all experimentally. It will thus be necessary to adopt the procedures mentioned earlier--that is, to combine experiments under a limited number of conditions with analysis and modelling to extrapolate the results to more general situations.

Some important combinations can be deduced from the types of members which have revealed the most distress during past earthquakes. For example, examination of buildings showing major damage or collapse indicates that the columns and the beam-column connections are the areas most likely to suffer damage.

Under seismic conditions columns are probably in a predominantly biaxial state with an axial compression somewhere between 0.2 and say .7 of the pure axial strength and with simultaneous shear and bending moment (which for simplicity will have to be assumed about one axis) sufficient, or almost sufficient, to cause failure.

Beam-column connections are subjected to much more complex state of stress and are probably best treated as a continuum acted upon by edge tractions which may have steep gradients. The high probability of large tensile stresses if of concern here owing to the low tension strength of plain concrete. Consideration of behavior under these conditions is probably inseparable from the problem of how to use plain concrete properties in modelling reinforced and prestressed concrete.

The lower parts of interior joints in structures moreover ought to be considered as carrying a compression of between about 0.2 and 0.7 of their pure axial strength combined with simultaneous compressions on the two axes orthogonal with the "axial" compression.

The range of possibilities is so large that work should be begun upon the problem immediately.

REVIEW OF PAST WORK: EXPERIMENTAL

This review will be brief; it covers mostly the period 1960-1976 and is not claimed to be fully comprehensive.

In 1964 Sinha, Gerstle, and Tulin [1] reported the results of uniaxial tests on three strengths of normal weight concrete. Their principal findings were:

(1) An "envelope" curve is the line which no stress strain curve exceeds, regardless of the loading path.

It was hypothesised that such an envelope curve exists for concrete under uniaxial compressive loadings.

The usual uniaxial stress strain curve is a good approximation for the "envelope" curve.

(2) In cyclic loading, in order to obtain an increment of deformation from one cycle of loading it is necessary to exceed a certain "critical" stress in the cycle.

(3) Unloading and reloading curves may be expressed as a family of curves. Fair approximations to the curves observed in their experiments were obtained by using parabolic expressions for the unloading curves and straight lines for the reloading curves. The parameters which make it of limited use in earthquake studies (the original study was not directed specifically toward earthquake conditions).

(a) The tests were uniaxial only.

(b) The loading rate was about 4 ksi/min (28 MPa/min) or about 30 $\mu\varepsilon$/sec.

A second study was undertaken by Karson in 1968 [2,3]. The relevant conclusions from this study include:

(1) Confirmation that the usual stress-strain curve approximates an envelope for uniaxial compression.

(2) In cyclic compression the points at which a reloading curve crosses the unloading cruve from the previous cycle may be called "common points". The location of these common points depends primarily upon the maximum stress and strain of the previous cycle. The minimum stress and strain of the previous cycle had little or no effect on the location of the common points.

(3) Predictions of cycles to failure for low-cycle high-stress conditions are presented. The limitations of this program include:

(a) Uniaxial compression only.

(b) Loading rates were between 2 ksi/min and 4 ksi/min or 1000 microstrain/min to 2000 microstrain/min.

The first study of conditions close to those occuring during earthquakes was conducted by Bresler and Bertero [4]. They tested 6" X 12" cylinders of 3 concretes--one of normal weight (NWC) and two of lightweight (LWC). The two different types of lightweight concrete were obtained by using two different aggregate sources.

One strength on NWC was employed (5 ksi = 34 MPa) and two strengths of each of the LWC (5 ksi and 3 ksi; 34 and 21 MPa).

In the first series of tests 257 6" X 12" (15 cm X 20 cm) cylinders were subjected to monotonic or cyclic loadings. In the second series 80 6" X 18" (15 by 45 cm) cylinders which contained spiral reinforcement were subjected to similar loadings. The second series thus simulated conditions in columns containing "confining" reinforcement.

The monotonic loadings were applied at rates between 10 and 100,000 microstrain/second while the cyclic loads were applied at about 20,000 microstrain/second. In cyclic loadings a minimum stress af about 0.1 f'_c was maintained while the maximum stress varied from about 0.5 to about 0.9 of the monotonic strength observed at the corresponding speed.

Constitutive relations were obtained for all tests. After about 20 cycles of loading the cycled specimens were retested to failure at a monotonic rate of 10 microstrain/second.

The principal conclusions were:

(1) The A.C.I. formula for Young's Modulus

$$E_c = 33\, w^{1.5}\, f'_c \text{ in psi units}$$

tends to overestimate the observed modulus; furthermore, this modulus is affected by the source of aggregate used in the concrete. the overestimations observed were as high as 30%.

(2) Strain at maximum compression, and total strain withstood before complete collapse, were sensitive to aggregate type and mix proportions.

(3) Increasing the strain rate from 10 to 100,000 microstrain/second produced moderate increases in strength and initial modulus.

(4) Cyclic stressing below about one half of the corresponding dynamic monotonic strength produced no change in the concrete.

(5) To obtain significant degradation of post-cycling strength and stiffness the peak stress during cycling must exceed about 85% of the dynamic strength. This conclusion applied at the experimental strain rate of 20,000 microstrain/second.

(6) If the cycling does exceed the level referred to in (5) then the post-cycling values of strength and stiffness are substantially reduced and large strains are accumulated.

(7) Confinement substantially improves the deformability of all concretes tested.

(8) Effectiveness of confinement in improving strength is much greater for NWC than LWC.

Table 1 summarizes some of the relevant research performed to date (1977) and points out areas where further studies are needed. The research needs are amplified in later sections of this paper.

REVIEW OF PAST WORK: ANALYTICAL

This will be a very brief outline of some suggested analytical procedures.

Monotonic Loadings

The earliest analyses used a simple linearly elastic model for plain concrete. Such a representation is clearly inadequate for cyclic loading or for high stress levels.

Nonlinear loading with elastic unloading under monotonic uniaxial compression is easily accomplished in a form suitable for computation by using one of many mathematical representations suggested in the literature. Most of these have been reviewed by Popovics [5]. Table 2 gives several suggested equations.

Multiaxial Loadings

A more realistic approach (based upon damage theory) has been suggested by Romstad, Taylor, and Herrmann [7]. In this representation biaxial states of loading are considered. The behavior of concrete is divided into "zones" each of which is supposed to represent a region of constant damage and which, therefore, has it own (elastic) modulus and Poisson's ratio. The zones are defined in two dimensional strain space (to avoid problems of non-uniqueness in stress space) and the properties to be used in an excursion from one point of the strain space to another are computed from the zone properties and the orientation of the change of strain during the excursion. The method was developed for monotonic biaxial compression with only one unloading permitted in its present form. The method could be modified for cyclic conditions.

The model parameters were estimated using the data of Kupfer, Hilsdorf, and Rusch [8]. Comparisons between experiments and predictions are given in Figures 1 and 2. Note that the data is used to "predict itself".

TABLE 1

AVAILABILITY OF INFORMATION ON CONSTITUTIVE BEHAVIOR OF CONCRETE UNDER SEISMIC CONDITIONS

Note: Numbers refer to References   E = *Experimental Data*   A = *Analytical Information*

CONDITIONS	NORMAL WEIGHT CONCRETE NWC	LIGHTWEIGHT CONCRETE LWC
1. Uniaxial Monotonic Compression to High Strain Levels	E: Well researched e.g. 1-5 A: see 2, 3, 5	E: 4 A: none
2. Uniaxial Compression at Seismic Strain Rate	E: 22 through 33 A: none	E: 21 A: none
3. Multiaxial Monotonic Compression to High Stress Levels	E: 48, 14, 15 * A: 8, 14, 15	E: 4, 19, 20 A: none
4. Cyclic Uniaxial Compression (Low Cycle High Stress)	E: 1 through 4 A: 1, 3, 10	E: 4 A: none
5. Cyclic Multiaxial Compression (Low Cycle High Stress)	E: 4 A: none	E: 4 A: none
6. Cyclic tests through several load paths in 3 space	E: none A: none	E: none A: none

* There are about 30 references in this area. See [12] for a partial listing.

TABLE 2 *

Equations suggested for Uniaxial Compression Stress-Strain Curve for Concrete

Author	Equation	Comment
For ascending branch only		
Bach	$\sigma = A\varepsilon^{n}$	Implies E infinite
Saenz	$\sigma = E\varepsilon[1 + (3E_\circ/E - 2)(\varepsilon/\varepsilon_\circ) + (1 - 2E_\circ/E)(\varepsilon/\varepsilon_\circ)^2]$	
Saenz	$\sigma = E\varepsilon/[1 + (E/E_\circ - 2)(\varepsilon/\varepsilon_\circ) + (\varepsilon/\varepsilon_\circ)^2]$	
European Concrete Committee	$\sigma = E\varepsilon(1 - \varepsilon/2\varepsilon_\circ)$	Implies $E/E_\circ = 2$
Sturman et al.	$\sigma = A\varepsilon(1 + B\varepsilon^{n-1})$	Implies $E/E_\circ = n/(n-1)$
Desai and Krishnan	$\sigma = E\varepsilon/[1 + (\varepsilon/\varepsilon_\circ)^{z}]$	Implies $E/E_\circ = 2$
Tulin and Gerstle	$\sigma = E\varepsilon/[A + (\varepsilon/\varepsilon_\circ)^{n}]$	Implies $E/E_\circ = A + 1$
Shah and Winter	$\sigma = E.\varepsilon.e^{-[(E\varepsilon^{-2})/A]^{m}}$	
Terzaghi	$\varepsilon = \sigma/E + A\sigma^{n}$	
Ros	$\varepsilon = \sigma/E + A\sigma/(B - \sigma)$	
For Complete Curve		
Smith and Young	$\sigma = E.\varepsilon.e^{-(\varepsilon/\varepsilon_\circ)}$	Implies $E/E_\circ = e$
Alexander	$\sigma = A\varepsilon/[(\varepsilon + B)^2 + C] - D\varepsilon$	

* Most of this Table is based on the review by Popovics [5]

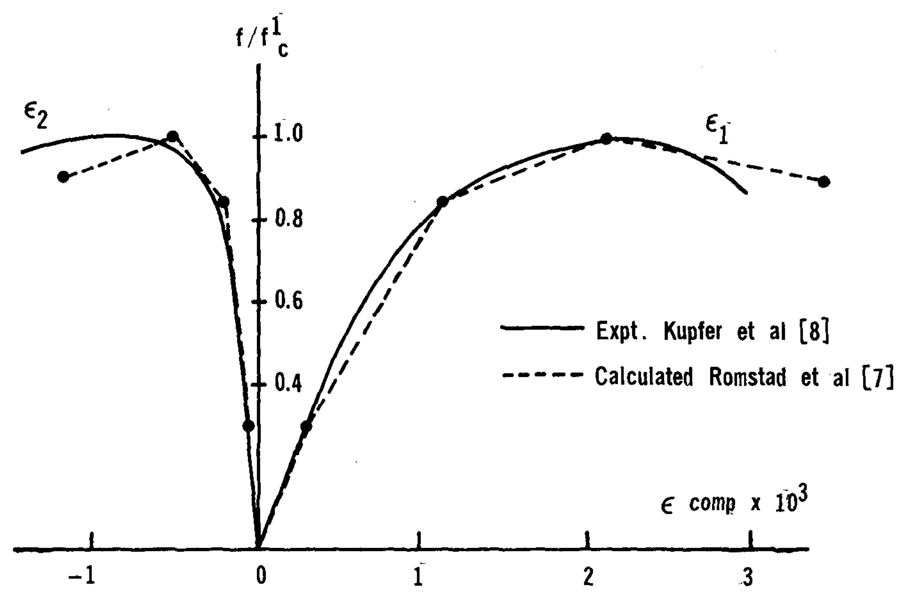

Figure 1. Predictions for Uniaxial Monotonic Compression Data of Kupfer et al [8] by Model of Romstad et al [7]

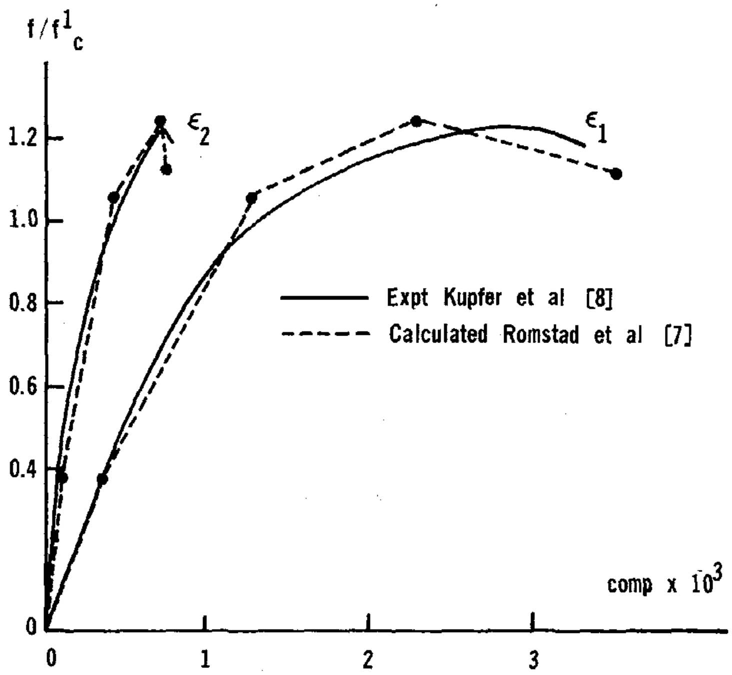

Figure 2. Predictions for Monotonic Biaxial Compression ($f_2/f_1 = 2$) Data of Kupfer et al [8] by Model of Romstad et al [7]

Cyclic Loading: Uniaxial

Sinha, Gerstle, and Tulin proposed a representation based upon their experimental program [9]. The envelope curve was approximated by a generalized second order polynomial plus a transformation constant thus calling for four constants to be obtained from experimental data.

Unloading curves were assumed to be parabolae; reloading curves to be straight lines.

Comparisons of analytical and experimental curves are shown in Figure 3. Note that the data used in the approximations came from the experimental curves, so this model is also "predicting itself".

Karson and Jirsa, extending the work of Karson, developed a more sophisticated model [3]. Their envelope* curve and curve of common* points were each described by a Smith and Young exponential expression (see Table 2).

The plastic (or permanent) strain appears to be a parabolic function of the strain at the common point for the cycle.

Unloading and loading curves are parabolae with the loading curves constrained to become tangent to the envelope curve. A computer program was written to perform the many necessary calculations.

Examples of computed and experimental uniaxial cyclic constitutive behavior are shown in Figure 4.

Karson's computational model was modified by Sharma and Bhattacharrya [10] to make it more efficient and to improve some of the representations. (For example, Karson's model predicted reloading parabolae which were convex from below whereas the experimental curves were convex from above).

The envelope* curve is a modified Smith-Young to improve the post peak representation.

The common point locus ("shakedown" points in Sharma and Bhattacharrya) is a modified Smith-Young curve.

Residual strains ("plastic" strain in Karson) are a parabolic function of common point strain.

Loading curves have two parts; a straight line up to the common point of the previous cycle (if the reloading begins below the common point) and a parabola above the common point which becomes tangent to the envelope.

Unloading curves depend upon the location of the point from which unloading begins. If unloading begins below the common curve then it is linear. If unloading begins above the common point the line is linear down to the common curve and is parabolic below the common curve.

Figures 5 and 6 show the improved predictions of Karsons data provided by this model.

*Defined in the section on experimental work.

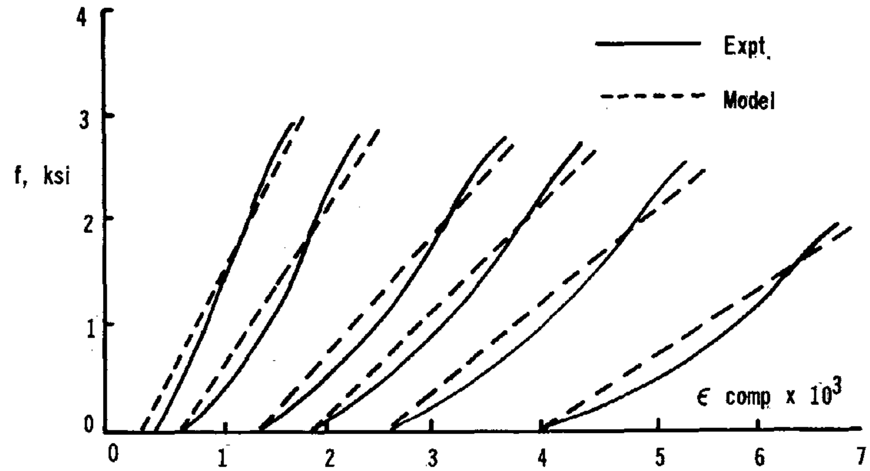

Figure 3(a) Reloading Curves

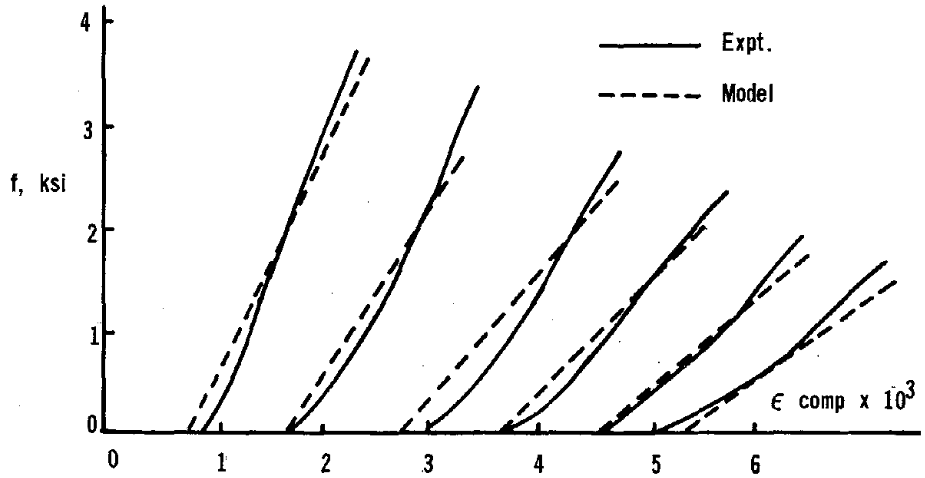

Figure 3(b) Reloading Curves

Figure 3. Model Predictions for Uniaxial Compressive Cyclic Loading for 4 ksi Concrete. Sinha et al [1]

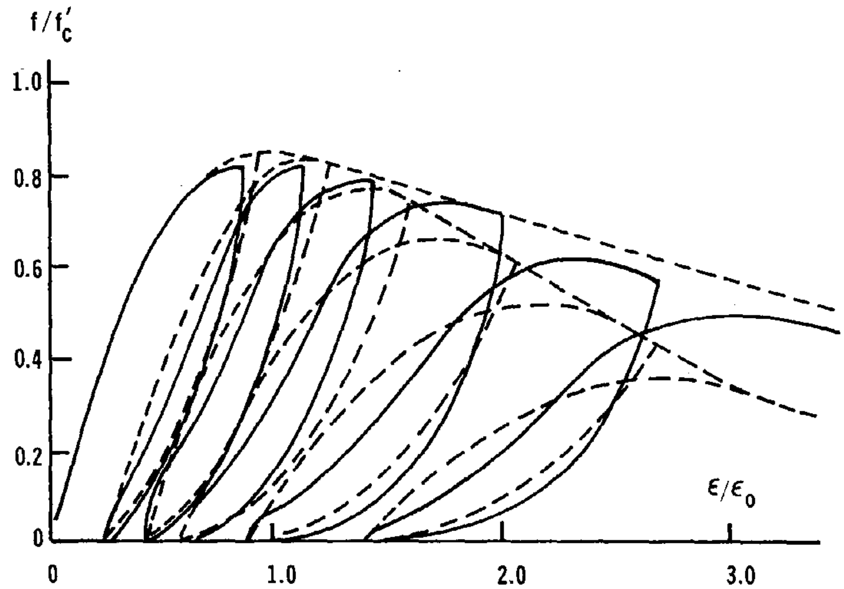

Figure 4. Model Predictions for Uniaxial Compressive Cyclic Loading From Karson [3]

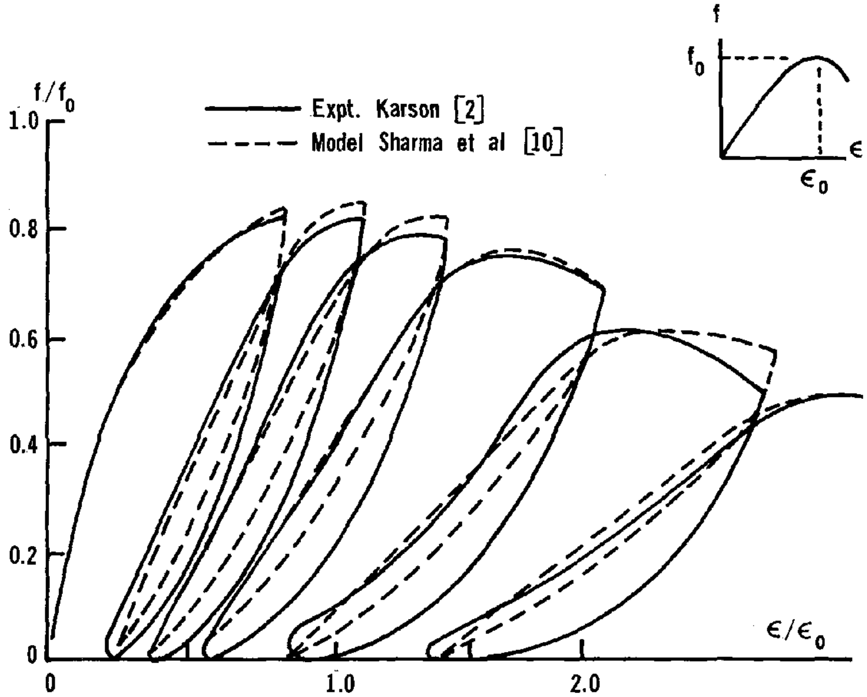

Figure 5. Predictions of Uniaxial Compressive Cyclic Loading of Karson [2] by Model of Sharma et al [10]

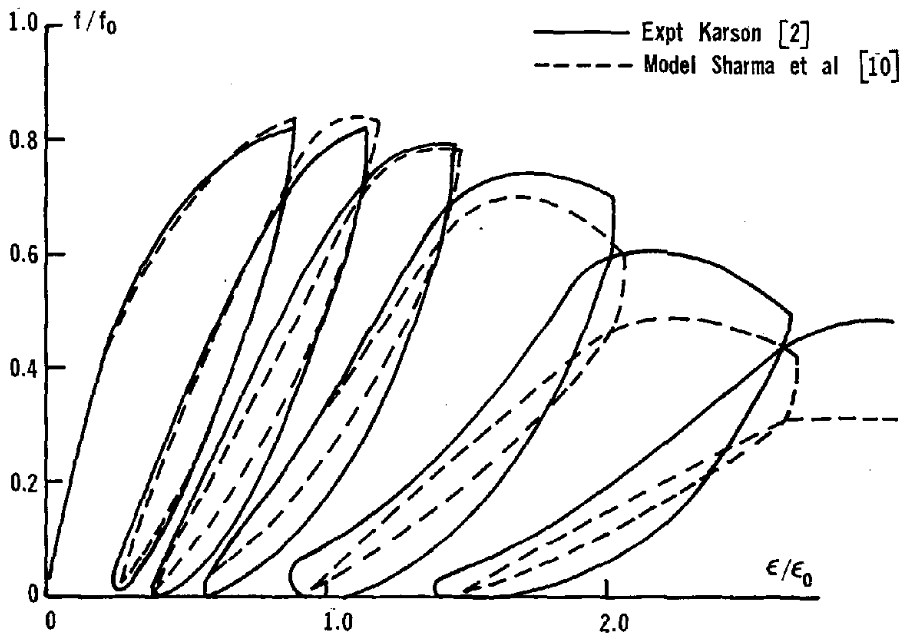

Figure 6. Predictions for Uniaxial Compressive Cycling Loading of Karson [2] by Model of Sharma et al [10]

Chen [11] has proposed a model for predicting the behavior of concrete under general multi-axial loading. This model is also based in large part on the biaxial (monotonic) compressive experiments of Kupfer, Hilsdorf, and Rusch [8].

Concrete is assumed to be a continuous, isotropic, linearly elastic then plastic, strain-hardening and fracturing material.

Three surfaces are defined in stress space. A failure surface; an initial discontinuity surface, which is defined as the limit of elasticity; and a "loading surface" which is defined as a surface which must be reached before additional permanent strains are accumulated during reloading.

Failure is treated separately in compression-compression and tension-compression regions.

The failure criteria are independent of the third invariant of the deviatoric stress tensor, and are assumed to be second order functions of the first invariant of the stress tensor and the second invariant of the deviatoric stress tensor.

The initial discontinuity surface and the loading surface have the same form as the failure surface.

The constants in these relationships must be determined from experiments. Figure 7 shows computed and experimental curves. Once again the model is "predicting itself".

Other models are described in references 12 through 17, and a discussion of several models is contained in [12].

RESEARCH NEEDS: EXPERIMENTAL

Preliminaries

A representative portion of a concrete structure will have principal stresses and strains which may be difficult to determine. Furthermore, some or all of the following may also be true:

(1) The principal stress directions and the principal strain directions may not coincide.

(2) All 6 quantities mentioned above may be oscillating in a random fashion.

(3) Material is discrete, quite probably non-isotropic, certainly non-linear, inelastic, and can crack (except possibly at very high levels of hydrostatic stress when it may "yield").

(4) Material is stochastic in nature. Large numbers of replications of each test are needed so that statistical statements may be made about the results.

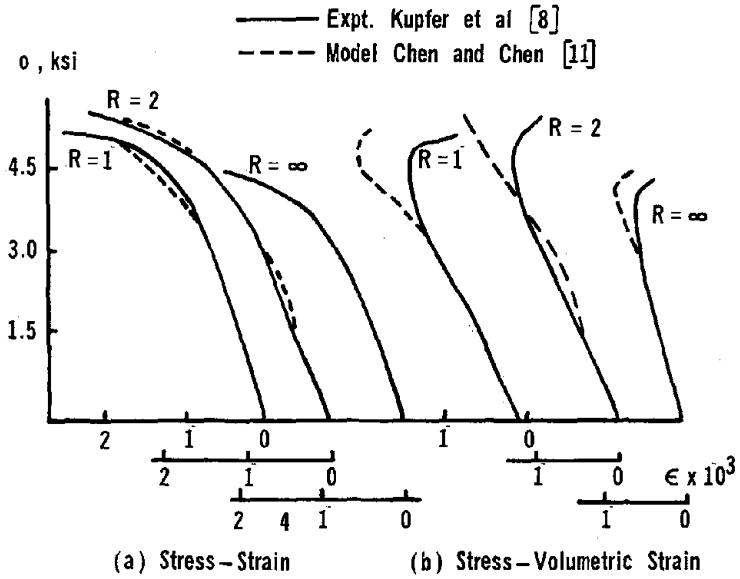

Figure 7. Predictions of Monotonic Biaxial Compressive Loading Data of Kupfer et al [8] by Model of Chen and Chen [11]

(5) Concrete properties vary with mix proportions, cements, aggregates, environmental conditions, etc. These properties, therefore, vary from place to place and time to time.

(6) In almost all structures the non-homogeneity and anisotropy are compounded by the presence of steel reinforcements.

(7) The concrete region may be NWC, LWC, or UHC or even an interface between two of these.

(8) Deformations may go up to or even beyond "failure" at rates up to about 30,000 microstrain/sec.

Faced with this situation the experimentalist cannot proceed without simplification. The usual procedure in such situations is to investigate the extremes of behavior and then attempt to interpolate. If the possible conditions are too many for a single experimental study one often uses intuition or a reasonable projection of behavior and conducts "spot check" tests to determine if further exploration is necessary.

Finally, just as the behavior of plain concrete is inseparable from that of reinforced concrete, so is it impossible to completely separate experimental and analytical needs.

In the recommendations which follow each suggestion is given a "Priority". Priority 1 is the highest priority.

A. Priority 1: Uniaxial Cyclic Behavior

Perform the following tests on a typical concrete.

	Experiment
(a) $\sigma_1 = \sigma_2 = 0$	σ_3 cycles from 0 to repeated failure at rates close to 30,000 $\mu\varepsilon$/sec
(b) $\sigma_1 = \sigma_2 = 0$	σ_3 cycles from 0.1 to 0.9 $f^1{}_c$ at the same rates
(c) $\sigma_1 = \sigma_2 = 0$	σ_3 cycles from 0.2 to 0.7 $f^1{}_c$ at the same rates

Reasons--This is basic information required by designers and those researchers of concrete who seek to understand its behavior. The test conditions also approximate the conditions in those portions of beams and columns remote from the connections.

B. Priority 1: Multiaxial Cyclic Behavior

Perform the following tests on a typical concrete.

(d) $\sigma_1 = \sigma_2 = 0.30\ f^1{}_c$ same cycling as (a), (b), (c)

(e) $\sigma_1 = \sigma_2 = 0.65\ f^1{}_c$ same cycling as (a), (b), (c)

(f) $\sigma_1 = 0 \quad \sigma_2 = .30\ f^1{}_c$ same cycling as (a), (b), (c)

(g) $\sigma_1 = 0 \quad \sigma_2 = .65\ f^1{}_c$ same cycling as (a), (b), (c)

Reasons--Observed vertical accelerations in buildings are frequently lower than horizontal values. Thus, the vertical load on a column may vary less than the moments in the girders which frame into it. If we simplify further to the case of unidirectional accelerations than a connection under such circumstances will have a stress state approximating the following: an axial load close to service value (say 0.3 $f^1{}_c$ when lightly loaded; 0.65 when heavily loaded) a transverse loading from the compression zone of the beam perpendicular to the motion which will have about the same range of values as the axial stress: and the stress from the compression zone of the beam parallel to the seismic action which will vary according to the intensity of the earthquake. etc. This explains cases (d) and (e).

Cases (f) and (g) will shed light on the influence of the intermediate stress which is of great interest in understanding the behavior of concrete.

C. Priority 2: New Materials

Repeat the tests of priority 1 on several different materials such as

(a) A range of strengths of NWC, e.g., 4, 5, and 6 ksi

(b) A range of strengths of LWC, e.g., 4, 5, and 6 ksi

(c) An ultra high strength concrete say 9 ksi

See also priorities 3 and 4.

Reasons--Much of our knowledge of concrete is based upon research performed many years ago with materials typical of those times. Today (1977) new materials are being used to a much greater extent. Designers are specifying higher strengths for NWC; lighweight concrete of 4 to 6 ksi strength (28 to 41 MPa) is becoming popular owing to the economies attendant upon the savings of weight; and consideration is now being given to ultra high strength concretes in the range 9-13 ksi (60 to 90 MPa) and to fiber and loop concrete. The present and future needs of the profession can thus be served only by testing such new materials prior to their widespread use.

D. Priority 2: Investigation of Variability

The testing of priorities 1 and 2 should be duplicated in two ways.

(a) By many replications (30 replications suggested)

(b) By duplication in other laboratories (2 or 3 in North America, one each in Central America, Europe, Japan, and South America suggested.

Reasons--Concrete is stochastic in nature. Variations in concrete properties of up to 30% are not unusual. This is often overlooked by designers computing moduli, stiffnesses, etc., with calculators capable of carrying several significant figures. The number of specimens normally used

(3 to 6 per test) makes it impossible to apply any statistical tests to the results and hence impossible to make any confidence statements about either the results or the range of values which a designer should use in his calculations.

Thus, 3(a) is needed to allow proper statistical treatment of the data.

The experimental results from tests on a specific concrete conducted at several locations will not always be the same [18]. The results are influenced by several factors including the testing machine characteristics, the type of loading platen and the technique of loading. Thus, tests are needed from several locations to examine and quantify the differences caused by these factors. Any locations could be used but there will clearly be greater interest in those areas with demonstrated interest and capability in such research and those where seismic events are of primary concern.

E. Priority 3: Tests of Damaged Concrete

Perform the following tests on selected concrete or concretes

(a) Stiffness and strength soon after (say few hours to a few days) being damaged by cyclic loading.

(b) Long time strength ("sustained" strength) and stiffness after being damaged by cyclic loading.

Reasons--Earthquakes are often followed by aftershocks. The behavior of surviving buildings immediately after a catastrophic earthquake is thus critical since

(i) overloads (e.g., from debris, etc.) may be present

(ii) people may be trapped, unconscious, dazed, or otherwise less able to fend for themselves

(iii) rescue services will be impaired.

This explains item (a).

Furthermore, one of the major post-disaster decisions is whether to replace or rehabilitate a structure. This explains item (b).

F. Priority 3: Repair of Damaged Concrete

Tests should be conducted to evaluate the efficiency of available repair methods, e.g., epoxy grouting and the effect of such treatment upon the constitutive properties of the concrete.

Reasons--Same as above.

G. Priority 4: Tests on Fiber or Hoop Concrete

Evaluate the expected improvements in ductility, toughness, and integrity offered by the new techniques of fiber and loop reinforcing.

Reasons--Plain concrete is brittle. Reinforced concrete is less brittle only because of the presence of steel bars which, however, are placed in very specific locations in accordance with specified loadings. The unexpected loads from an earthquake often point out the brittle portions of a structure. It has been suggested that the use of many small reinforcements throughout the material instead of a few discrete bars may transform the concrete into a more homogeneous and ductile material with greater energy absorbing capacity.

H. Priority 4: Tests of Aggregate Properties

Evaluate

(a) The effect of aggregate type and properties upon stiffness of concrete, especially the range of values to be expected.

(b) The effect of aggregate type and properties upon the tests of priorities 1 and 2.

Reasons--Evidence indicates that the aggregate has large influence upon stiffness of concrete. This is of importance in computing the dynamic characteristics of a structure. The range of values to be expected will assist the designer in selecting the range of forces to which the structure may be subjected. Hence, item (a).

Concrete is a mixture. And while the properties of one concrete may be similar to those of another there is no guarantee of this. Hence, since obtaining the information of tests of priorities 1 and 2 will be a major undertaking it is likely that it will be done, initially, for only one or two concretes. Item (b) will thus be needed to ensure that concretes using different aggregates (e.g., from another country) will not be too different from the test concrete.

I. Priority 4: Tests With Two Cyclic Stresses

Perform the following tests:

Test

(a) $\sigma_3 = 0$ or small value; cycle σ_2 and σ_3 from .3 to .7 $f^1{}_c$ in synchronisation.

(b) $\sigma_3 = 0$; cycle σ_2 and σ_3 from .3 to .7 $f^1{}_c$ but 180° out of phase.

Reasons--Earthquakes rarely occur exactly parallel to one axis of a structure. Hence, in a connection, both beams may be expected to have oscillating stresses. Items (a) and (b) simulate the extremes of load paths under these conditions.

RESEARCH NEEDS: ANALYTICAL

A. Priority 1: Formulation of Models to Predict the Constitutive Behavior of Concrete Under General Cyclic Loadings

The spectrum of all possible histories of loading during a seismic event is infinte in extent. Thus, it will be impossible to conduct a test (or series of tests) for every possible sequence of loading. Consequently, it is necessary to formulate a model which can predict the behavior under completely general cyclic loadings. Unfortunately, most attempts in the past have either been too qualitative rather than quantitative or else have been used only to predict the data from which the model parameters were determined (i.e., used "to predict themselves" in a sense). Some candidates are:

(a) Cumulative damage theories

(b) Fracture mechanics theories

(c) physical models (e.g., aggregates discs in mortar matrix)

(d) Clastic mechanics (see for, e.g., Staff and Zienkiewicz [ ])

(e) "Classical" theories (plasticity, endochronic theories, etc.)

B. Priority 1: Data to Evaluate Proposed Models for the Constitutive Behavior of Concrete

No detailed proposals are made for such experiments at present.

The number of possible stress (or deformation) paths which concrete may experience during an earthquake are infinite. It will, therefore, be imperative to have a model, preferably physically based, which can compute predictions of the constitutive behavior for any path in three-space. Many models have been suggested (for descriptions see, for example, Taylor [12] or Popovics [5]) but usually these predict data from models whose parameters were based on the very data being predicted. To paraphrase Eddington we need a model which is vulnerable (to testing). Thus, this is not a purely experimental problem but must be considered in conjunction with analysts. A model must be formulated so that its suitablility is open to test. In particular, it is very desirable that there be a test (or tests) which can permit comparisons of the efficiencies of several suggested models. The development of such models is given the highest priority in the recommendations for analytical work.

C. Priority 2: Application of Laboratory Studies of Plain Concrete to Real Structures

Considering that

(a) Concrete tested uniaxially in the laboratory shows different strength and peak strain than the same concrete tested in flexure, i.e., with a strain gradient.

(b) Tests on the same concrete but in different testing machines may give different results.

(c) Tests on the same concrete but with different platens may give different results.

(d) Tests of the same concrete but with applied loads versus applied deformations may give different results.

(e) Tests on the same concrete but with different sizes of specimens may give different results.

(f) Tests on virgin samples often gives different results from samples which have been pre-loaded a few times.

(g) Tests on cylinders versus cores give differences which are still not clearly explainable quantitatively.

it is quite apparent that there are many questions to be answered before the data from laboratories can be used directly in design calculations. The resolution of these questions will also straddle the experimental and analytical fields. Information on all these topics is available but is sparse. A major effort is needed to gather such data; organize it into a logical format; identify gaps where further research and/or analysis are needed; and to present suggestions for modifying laboratory data for specific field conditions if this appears necessary.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
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CONFINED CONCRETE: RESEARCH AND DEVELOPMENT NEEDS

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INTRODUCTION

Efficient seismic-resistant structures require structural material that is lightweight, stiff, strong, and tough under generalized types of excitations. In other words, the structural material should not only have high ratios for maximum strength to unit weight and modulus of elasticity to unit weight, but it should also be capable of absorbing and dissipating large amounts of energy under repeated actions, including reversals of deformation. These mechanical characteristics are not inherent in the case of "plain" concrete, except when subjected to compressive hydrostatic pressure, and research is needed to improve its deformability, particularly in tension where plain concrete is very weak [1].

In his state-of-the-art report on "Constitutive Relations for Concrete under Seismic Conditions" [2], Taylor points out that

> plain concrete in its present form may not be the best energy-absorbing material for structures in seismic regions...A study of the properties of new materials such as fiber and loop concrete may permit cost-benefit analysis to decide this question.

However, when properly "confined," plain concrete can be converted into a tougher, more ductile structural material. An ideal confinement would be one which can permit the development of stress states equivalent to compressive hydrostatic pressure.

Confinement of flexural structural members (beam-columns) and strut elements is generally achieved through proper design and detailing of lateral reinforcement. The possibility of prestressing and/or using expansive (self-stressing) types of concrete to induce states of stress equivalent to compressive hydraulic pressure should be investigated.

Effects of Lateral Reinforcement

Ever since Considere published his findings on reinforced concrete in 1903 [3], designers have recognized the advantages of using lateral reinforcement to enhance the strength capacity of structures. Even today, however, the complete load-deformation behavior of confined concrete under stresses and strains that can be developed under earthquake excitations is not very well understood. As Taylor indicated [2], the cyclic multiaxial behavior of a nonlinear, strain-softening, discrete, cracking material such as concrete is so complex that it is difficult to carry out comprehensive experimental investigations which cover all possible combinations of special conditions that can exist during a structure's response to seismic motions.

Bažant, in a recent paper [4], also discusses the difficulties involved in formulating general constitutive relations for concrete. He mentions several unusual concepts and factors which must be considered, such as: (1) the sensitivity of intrinsic time increments (measuring the accumulation of inelastic strain) to hydrostatic pressure; (2) inelastic dilatancy due to shear straining; (3) description of strain-softening tendency; (4) dependence of tangent moduli on dilatancy (not on stress or strain tensor); and, if long time nonlinear creep should be modeled, also (5) introduction of more than one intrinsic time.

At present, data available on the behavior of confined concrete under seismic excitations are not only sparse but of a very limited nature. In view of the complexity of the problem, the most immediate remedy is to try to define the bounds on the expected behavior and to conduct some comprehensive studies on these bounds. Therefore, designers and analysts must devise methods for interpolating the data to other circumstances, which is not an easy task.

The ideal would be to formulate a unique general stress-strain relationship for concrete, although this does not seem possible in the near future. Even if such a relationship could be formulated, its complexity might hinder understanding of the basic physical mechanisms involved, possibly leading to serious errors or a false sense of achievement. Researchers, designers, and analysts must devise the simplest possible analytical model capable of producing reliable results for the problem at hand. For example, if the designer is only interested in predicting the usable strength of a standard flexural member, an effective rectangular stress-strain relation based on Hognestad's curve [5] could be used. On the other hand, if information is desired on the deformational capacity of a critical region of the member, a more refined stress-strain curve accounting for the effect of confinement should be used, with the model to be used determined by the type of excitation --monotonically increased or cycled with or without reversals. As pointed out by Darwin and Pecknold [6], "...the realism necessary in the constitutive model depends on the structural type, the response quantity sought and whether the load is monotonically increased or cycled as would occur in an earthquake for example."

To formulate these simple realistic constitutive models, designers and analysts must have a clear understanding of the physical mechanisms controlling the behavior of actual materials under the particular state of stress or strain being considered. Research efforts should therefore be devoted to this end. For example, in the case of shear walls, it is necessary to consider the behavior of reinforced concrete under biaxial stresses. Only recently a nonlinear constitutive model for plain concrete subjected to cyclic biaxial stresses was proposed and is being used to predict behavior of shear panels previously tested [6]. No successful application to cyclic loading (including stress or strain reversals) of reinforced concrete structures under multiaxial states of stress have been reported, although the extension of the endochronic theory of inelasticity and failure to concrete appears very attractive [4,7].

Objectives and Scope

The main objectives of this paper are to review present knowledge on behavior of confined concrete, to evaluate present constitutive relations, and to discuss research needs in this area. Emphasis is placed on behavior under seismic excitations.

After briefly discussing the difference between confined and plain concrete, the paper reviews present knowledge on the behavior of confined concrete by examining past experimental and analytical studies on this subject and discussing the constitutive models that have been suggested. Only those studies that have not been discussed by Taylor in his state-of-the-art report [2] will be reviewed herein. The application of these models in predicting the main parameters of seismic response is also discussed and evaluated. From the review and evaluation, research and development needs to improve knowledge of the properties and behavior of confined concrete are suggested and ways of formulating more realistic constitutive relations for this concrete are offered.

CONFINED CONCRETE

Reinforced concrete flexural elements are usually laterally reinforced. Because reinforcing steel must be protected against corrosion, fire etc., in each region of an element two different types of concrete must be distinguished: "plain" concrete, which forms the protective cover or shell of the element, and "confined" concrete which forms its core, that is, the concrete within the cover which is restrained by the lateral reinforcement. The behavior of the confined core is essentially different from that of the unconfined cover or plain concrete. Although these two types of concrete at their contact undergo similar deformations under loading, the two adhere to different stress paths. Furthermore, the concrete cover of the confined core spalls at longitudinal strains smaller than those observed from tests conducted on plain concrete specimens. The closer the spacing and the larger the size of lateral reinforcement, the less will be the effectiveness of the cover in resisting loads [8]. There is a need for improving understanding of the behavior and effectiveness of concrete cover with regards to the tie size, tie spacing, cover thickness, etc.

Unfortunately, what constitutes cover and confined core in an actual reinforced concrete structural element is unclear due to the arch formation between ties. The difficulties involved in defining these two parts at any cross section of a laterally reinforced concrete element have been discussed by Sargin [8] and Park and Paulay [9]. It is therefore not surprising that among researchers, discrepancies often result regarding the actual load-deformation behavior of confined concrete. A constitutive relation for confined concrete formulated by one researcher is rarely the same as that formulated by another since the relation is usually applicable only for the test condition from which it was derived.

REVIEW OF PAST STUDIES ON CONFINED CONCRETE

This review complements that carried out by Taylor in his state-of-the-art report [2].

An extensive review of work on the stress-strain relationship of confined concrete under uniaxial compression carried out up to 1970 has been done by Sargin [8]. Most of the studies of confined concrete of realistic reinforced concrete structural elements were carried out after 1961 and grew out of the concept of "ductile concrete" introduced by Blume, et al. [10].

In 1971 Burdette and Hilsdorf [11] published the results of their investigation on the behavior of laterally reinforced concrete columns. Park and Paulay [9] have reviewed the studies conducted on confined concrete up to 1973. Since then, several other works have been carried out, some experimentally [12-16] and others analytically oriented [4,7,17,18].

In reviewing the results of all these studies it is convenient to distinguish the case of confined concrete under external uniaxial compression from those of confined concrete under external multiaxial states of stress.

Stress-Strain Relationship for Confined Concrete under External, Monotonically Increasing Uniaxial Compression

In general, a designer of seismic-resistant structures is interested in three main mechanical characteristics of the structural material: initial stiffness, yielding and/or maximum strength, and deformational capacity (ductility). From the results obtained there is general agreement among researchers that the use of lateral reinforcement which meets or exceeds present seismic code requirements for lateral confinement causes: (1) no significant change in the initial stiffness of the corresponding plain concrete; and (2) an increase in concrete ductility (deformability or toughness), that is, the strain at which the strength of confined concrete starts to decrease significantly is considerably higher than that observed from the results obtained on plain (unconfined) concrete. However, there is no complete agreement regarding the specific quantification of the observed increase in ductility. Regarding strength, the lack of agreement is even greater.

Although no complete explanation for the above lack of agreement can be offered, the authors believe that part of the problem concerns the way in which stress-strain relationships were measured in the different experiments conducted and the manner in which the stress and strain were computed from the measurements obtained. The main problem is that stress and strain cannot be measured at a point, and while stresses, f_c, are usually obtained from measurements of the loads, P, and areas, A, where $f_c = P/A$, the strains, ε, are computed from the measured elongation, ΔL, along certain lengths, L, where $\varepsilon = \Delta L/L$. Therefore, what is usually represented are average stresses versus average strains, which implies that the derived stress-strain relationship includes many particular test and measurement conditions.

To compare the different relationships, it is necessary to reduce them to a level where they are a function of the basic material characteristics, rather than the way in which they are measured. This requires determination of the true stresses and true strains of the confined concrete. The need for this kind of data reduction is shown in Fig. 1, which plots and compares the stress-strain relationships obtained using different types of instrumentation and different ways of computing the average stresses and strains. This figure indicates that the actual stresses in the core can be much higher than the stresses calculated over the gross area, A_g, or even higher than the stresses calculated on the basis of a constant defined core area, A_c, generally defined in codes as that area which is enclosed by the outer perimeter of the confining hoops. Similarly, the actual strains in the core can be significantly higher than the measured average strains. Sargin [8] has developed equations that

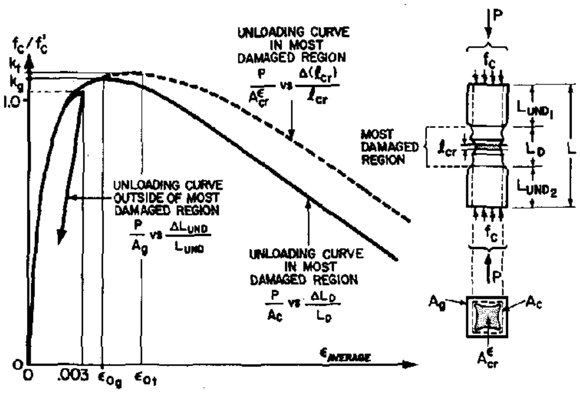

FIG. 1 COMPARISON OF DIFFERENT STRESS-STRAIN RELATIONSHIPS THAT CAN BE OBTAINED DURING TEST OF ONE CONFINED CONCRETE SPECIMEN

permit the effective core area to be computed and also discusses how the non-uniform strain distribution along the entire length of a specimen can be included in computing its total deformation.

As discussed above, it is not possible at present to obtain actual stress and strain values. However, attempts should be made to measure the variations of the area of the critical section as the test progresses, as well as to use the smallest gage length possible in measuring the deformation of the fibers at the critical section.

The effects of different measurements and definitions of stresses and strains in the derivation of the constitutive model for confined concrete have been discussed in reference 13. In this study it was evident that to predict the behavior of any reinforced concrete specimen, not just those that have been tested, it is necessary to obtain the relationship between the true stresses and true strains rather than between the average values. Furthermore, it is well known that the average stress-strain relationships of confined concrete are sensitive to the type and arrangement of the lateral reinforcement.

Confinement due to rectangular ties--Of all the idealized, monotonically increasing stress-strain relationships that have been suggested to date, the most commonly used is that proposed by Kent and Park [19]. It should be noted that this stress-strain idealization has been derived for confinement using simple arrangements of rectangular hoops (ties) and that there has been very little testing of concrete confined by the more complex arrangements of transverse reinforcement typical of columns in practice [20]. Some of these hoop arrangements were investigated and reported in references 13 and 14. The arrangement of lateral reinforcement used in the studies reported in reference 13 is illustrated in Fig. 2. Besides studying the degree of confinement offered to plain concrete by the type of lateral reinforcement shown in Fig. 2, the effect of concrete cover and longitudinal reinforcement were also investigated.

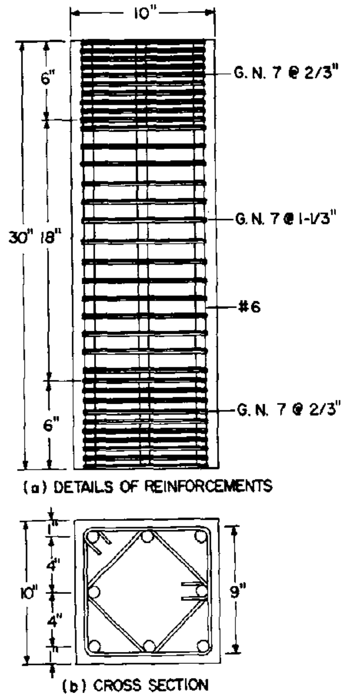

FIG. 2 DETAILS OF SPECIMEN REINFORCEMENTS (IN. = 25.4 MM) [13]

From the results obtained and presented in reference 13, the following conclusions were drawn.

1. Although the confined concrete specimens showed a definite increase in the maximum concrete stress over that of the unconfined concrete specimens, the main increase was in the ductility capacity. While the increase in strength was only 13%, the increase in ductility was up to 300% at the point where the maximum strength was reached and up to 1000% at the point of the descending branch of the curve where $f_c/f'_c = 0.85$.

2. For the specimens tested, the values of the confinement effectiveness coefficient, k_o,* at the maximum stress level ranged from 1.35 to 2.07, while the UBC and ACI codes specify 3.33 and 1.88, respectively. The definition of k_o, as well as its significance for the design of columns, is discussed in detail in reference 12.

3. By improving the basketing of the concrete through the addition of longitudinal reinforcement, the confinement of the concrete was correspondingly improved, resulting on an average of a 7% increase in the maximum strength of concrete stress. Initiation of buckling of longitudinal reinforcement was detected for longitudinal strains of about 0.025. Since the decrease in load-carrying capacity of the steel due to buckling was not considered in determining the stress-strain relationship of the confined concrete, the relation presented (Fig. 3) is conservative for strains beyond initiation of buckling of the longitudinal reinforcement.

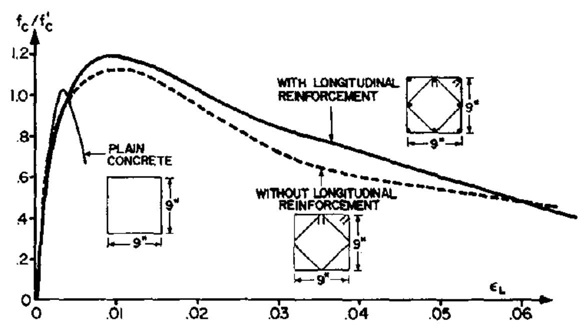

FIG. 3 REINFORCED CONCRETE SPECIMENS EXCLUDING COVER--WITH AND WITHOUT LONGITUDINAL REINFORCEMENT (IN. = 25.4 MM) [13]

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*$f^{conf.}_{c\ max} = f^{unconf.}_{c\ max} + k_o f_r$, where f_r is the confinement pressure.

4. No abrupt drop was observed in load resistance of the specimen when the concrete cover started to spall. The spalling of the cover appeared to pull away a larger volume of the supposedly confined concrete between two consecutive ties than that which spalled due to arch formation in the specimens without cover. To consider the effect of the crushing and spalling of the concrete cover on the effective area of concrete resisting axial compression, a linear variation of the area from 1.0 A_g at $\varepsilon_L = 0.0025$ to $(A_c/A_g)A_g$ at $\varepsilon_L = 0.01$ was a reasonable approximation for the specimens tested (Fig. 4).

5. None of the current methods (constitutive models) for predicting behavior of axially loaded confined concrete were capable of predicting accurately the complete stress-strain relationship obtained in the tests (Fig. 5). While some of them underestimated the maximum stress and/or strain at which this maximum stress was reached, other models underestimated the strain values in the descending branch of the stress-strain curve.

6. An improved stress-strain relationship is suggested which considers the increase in strength and ductility caused by the type of lateral and longitudinal reinforcement used in this study and offers better agreement with the experimental results obtained than those predicted using current constitutive laws (Fig. 6).

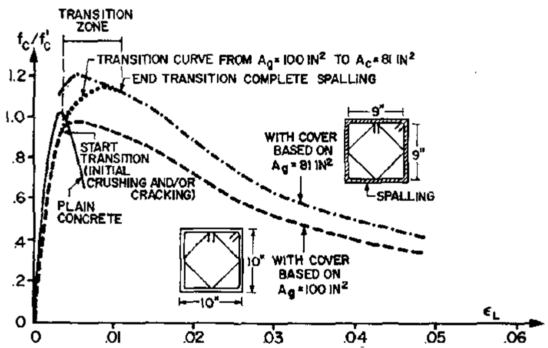

FIG. 4 COMPARISON OF GROSS AND CORE STRESS-STRAIN RELATIONSHIP (IN.=25.4 MM)[13]

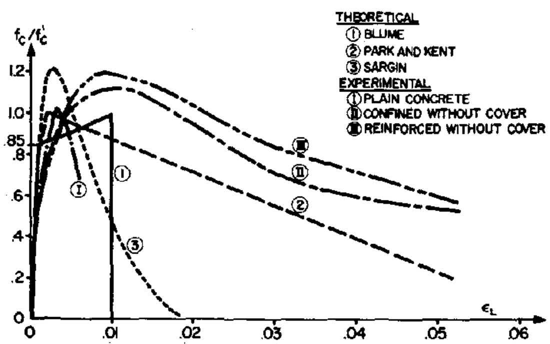

FIG. 5 COMPARISON OF ANALYTICAL CURVES WITH EXPERIMENTAL RESULTS [13]

Concrete confined by circular hoops--There are less experimental data available on circular sections confined by circular hoops or spiral reinforcement than those for rectangular ties. Leslie [21] has postulated a stress-strain curve for circular confined concrete columns, similar to that developed by Kent and Park [19] for rectangular confined concrete. Tests have shown that the mechanism of confinement for circular and rectangular hoops are quite different [8,9,11,16]. Circular hoops (welded or in the form of a continuous spiral) are the most effective form of lateral reinforcement for restraining transverse expansion. Hence, the stress-strain curve for circular sections confined by circular hoops

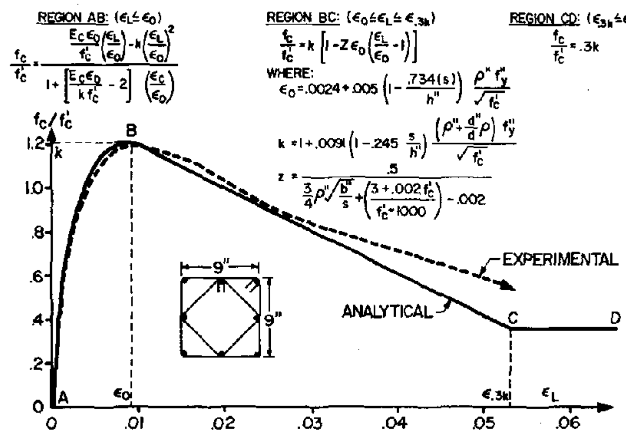

FIG. 6 CONFINED CONCRETE WITH LONGITUDINAL REINFORCEMENT--ANALYTICAL CURVE AND ITS COMPARISON WITH EXPERIMENTAL RESULTS (IN. = 25.4 MM) [13]

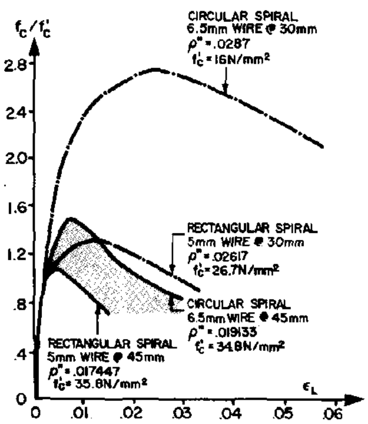

FIG. 7 COMPARISON OF STRESS-STRAIN RELATIONSHIPS FOR CONCRETE CONFINED WITH CIRCULAR AND RECTANGULAR SPIRALS [22]

can be expected to differ from those confined by rectangular hoops. The greater effectiveness of circular confinement should be reflected not only in the larger increase in the strength of the confined concrete but also in the characteristics of the descending branch as well as in the damage pattern and mode of failure. Analyses of the results available [10,12,21,22] confirms the above. Figure 7 illustrates the difference between the stress-strain curve for concrete confined with either circular or rectangular spirals.

Effect of Strain Gradient on Flexural Members

In most structures the critical reinforced concrete regions are subjected to strain gradients, and the applicability of constitutive models derived from concentric compression tests under these conditions has been questioned. Although the effects of strain gradient on concrete behavior are not completely known, there are indications that the values of the modulus of elasticity, E_c, the compressive strain corresponding to maximum stress, ε_o, and the ratio of maximum stress to cylinder strength, $f_{c_{max}}/f'_c$, increases with increase in strain gradients. Sargin [8] has proposed some equations which quantify the effects of strain gradients. However, such equations are derived from limited data and can be considered of an exploratory nature only.

A series of tests has recently been carried out at the Portland Cement Association laboratory [15] to determine the behavior of confined concrete under strain gradients. The results obtained have been presented as stress-strain curves in which the stresses were computed on the basis of the gross area. This makes it difficult to interpret the actual stress-strain that developed in the confined concrete. Comprehensive investigations of the effect of strain gradient in confined concrete should be conducted since this information is essential in studying in detail the behavior of critical regions of flexural structural members.

Behavior of Confined Concrete (Stress-Strain Relationship) under Seismic Excitations

As pointed out by Taylor [2] seismic disturbances of a structure create special conditions which are not normally present in structures subjected to "standard" types of excitations. Behavior of concrete in the domain defined by these conditions has only been studied very recently, and such studies are few in number and limited in scope. Although some studies on the effect of strain rate on confined concrete have been conducted [12], more comprehensive studies on the effect of this parameter are needed.

Most studies have been conducted under uniaxial compression. For this case, researchers usually assume that the stress-strain relationship for confined concrete under cyclic loading is represented by a family of unloading and loading curves whose envelope is the curve obtained under monotonically increasing uniaxial compression. In idealizing hysteretic curves, different investigators have made different assumptions [6,7,23,24].

Despite different idealizations of the hysteretic curves, all the researchers obtained good agreement with their experimental results. Perhaps one reason for this is that these models were all used to predict flexural behavior of very ductile members which are not highly sensitive to the nonlinear behavior of concrete [24]. This may not be true for other structural members whose behavior can be sensitive to the cyclic behavior of concrete. This will be discussed later in the case of shear wall panels.

The authors believe that knowledge of the actual hysteretic behavior of concrete--unconfined and confined--is of particular importance in the case of structures that have been subjected to damage and are then repaired, particularly, if cracks are filled with epoxy or any other material. The subsequent response of the structure, even under service loads, can be very sensitive to the previous hysteretic behavior of this repaired concrete. Thus, efforts should be devoted to improving knowledge of hysteretic behavior at large inelastic strains.

Effect of Transverse Shear on Hysteretic Behavior of Flexural Members

Most of the investigators that have worked on the problem of predicting the hysteretic behavior of reinforced concrete members or structures under seismic conditions have based their predictions on existing bending theory in which only curvature due to normal flexural strains are considered. While excellent agreement has been obtained in predicting response under monotonically increasing deformation, and good correlation has been obtained up to the end of the first cycle involving full reversals of significant inelastic deformation little agreement has been obtained for subsequent cycles. The larger the shear stresses, the lower is the correlation.

The presence of shear forces induces shear deformations which contribute to the deflection of the elements. Furthermore, inelastic shear strains produce volume dilatancy which, in turn, induces significant forces in the stirrups. These forces initiate normal stresses in the concrete, converting the plain concrete into confined concrete. These two effects have been modeled by Bažant [7] who has applied the endochronic theory of inelasticity to plain concrete. The difficulty in applying this or any other theory to predict the effect of shear is with regards to predicting crack formation and propagation, crack closing and reopening. Bažant assumes that there is only one crack system in which the cracks are open and accounts for this by changing the original matrix using a shear transfer factor, α, that was originally introduced by Suidan and Schnobrich [25]. This shear transfer factor represents the effect of aggregate interlocking on rough surfaces of opened cracks. Although Bažant recognizes that this factor decreases with the opening of the crack and increases with the relative displacement parallel in the crack, following the suggestions made in reference 25, he assumes a constant value of $\alpha = 0.5$ in his numerical calculations. Although application of this assumption yielded satisfactory agreement with test data, it is questionable whether this assumption can be applied to predicting behavior of real elements under high shears.

The complexity of predicting the effect of shear in the hysteretic behavior of flexural members subjected to reversals of deformation has been discussed by Ma, Bertero, and Popov [24]. After discussing the basic mechanism of shear transfer and resistance in cracked regions subjected to load-deformation reversals, these authors formulated a mathematical model for predicting the hysteretic shear force-deformation relationship. Emphasis was placed on considering analytically the observed increase in shear resistance degradation with increases in both the magnitude of the applied shear and/or deformation and in the number of cycles of reversals. The possible shear degradation mechanisms include (1) opening of cracks due to yielding or slippage of the main reinforcement; (2) spalling of the concrete cover around the periphery of the flexural critical region; (3) degradation in stirrup-tie anchorage due to large variations in the strains where it is crossed by inclined cracks, and/or by splitting and spalling of the concrete cover; (4) crushing and grinding of concrete at the crack surfaces which could lead to less effective aggregate interlocking resistance along the open cracks; and (5) local disruption of bond between the longitudinal steel and concrete due to dowel action along the open cracks.

The analytical model used offered reasonable prediction of the shear degradation which occurred at the initial loading of reversals at a displacement ductility ratio of one, and in the first cycle at a ductility ratio of two. It could not predict the shear degradation occurring during reversals at a displacement ductility of two. This is because the model used for the shear resistance elements, i.e., aggregate interlocking, tie resistance across the cracks, and dowel action of the main bars, did not account for the effect of their degradation due to reversals. It seems that in order to predict shear degradation (particularly, at the initial phase of reloading) due to repeated reversals at a displacement ductility of two or greater, it is essential to incorporate into the analysis a degrading model for the elements resisting shear along the large open cracks. More specifically, it is essential to obtain data regarding the hysteretic behavior of all such resisting elements. The analytical results indicate that it is necessary to formulate a degrading aggregate interlocking resistance model to predict the initial shear stiffness degradation that occurs under loading reversals. It is also necessary to have better information regarding gaps that can be developed between both the ties and main bars and the main bars and confined concrete.

Since the shear force-deformation response was predicted on the basis of the observed crack pattern, measured crack width, and yielded length of the main steel, it would be desirable to predict these parameters analytically. To do this requires knowledge of the mechanisms controlling the interacting inelastic shear and flexural deformations as well as the mechanisms of slippage of the main bars.

Two observations, whose importance cannot be overemphasized, can be drawn from the above description. First, the problem of shear effects in flexural members cannot be isolated from the problem of flexural deformation. Secondly, it must be recognized that reinforced concrete is a composite material, not two independent materials. Thus, understanding of the hysteretic behavior of reinforced concrete structures under seismic excitations necessitates knowledge of the interaction of the two constituent materials of this composite material. Independent development of better constitutive models for each of the two constituents (plain concrete and reinforcing steel) although necessary, is not sufficient. Development of constitutive models requires careful study of the interaction between these two materials. Among the main parameters that control the hysteretic behavior of reinforced concrete elements is the bond, or interaction between steel and concrete [26].

Effectiveness of Steel-Concrete Interaction under Seismic Excitations

Experiments have shown [24,26] that the effectiveness of stress transfer between steel and concrete under monotonically increasing loads is quite different from that under cyclic loads which induce deformational reversals. When the intensity of the stress induced in the steel bar due to this type of cyclic load exceeds certain values, the slippage of the bar increases with each cycle. Although some bond stress-slip relationships have been formulated and used [24,26], they are accurate only when used to predict behavior of reinforced concrete elements similar to those that have been used in the tests from which the data used in the derivation of the relationships have been obtained.

Bond deterioration in the anchorage and splicing of reinforcing bars is one of the weakest construction links in seismic-resistant reinforced concrete structures that have been designed according to recently recommended or proposed seismic codes. At present, the amount and spacing of lateral reinforcement required to provide adequate (1) confinement of concrete, (2) lateral restraint to the main bars to delay their buckling, and (3) resistance to the shear effects, are such that most of the failure mechanisms (at the columns and beams, and at the joints) that have been observed in buildings designed according to previous codes will be suppressed in buildings designed according to recent codes. Instead, failure mechanisms will likely develop due to large slippage of continuous bars along their embedment length at interior joints and/or at their end anchorage, particularly if lapped splices are used.

Thus, it is important to improve our knowledge of the stress transfer between steel and concrete under seismic excitations and to search for a general bond-slip relationship which will permit realistic seismic response analyses of reinforced concrete structures to be carried out.

Concluding Remarks Regarding Constitutive Models of Confined and Unconfined Concrete to Predict Seismic Behavior of Flexural Members (Beams and Columns)

Although a number of constitutive models for concrete has been formulated, development of a general model for the detailed analysis of the hysteretic behavior of flexural, reinforced concrete members under seismic excitations requires further study. Integrated experimental and analytical studies are necessary. Before a general model can be formulated, better understanding is needed of the basic mechanisms controlling the behavior of the reinforced concrete composite material in the critical regions of the flexural member.

Constitutive Models of Concrete under External Multiaxial Stresses Induced by Seismic Excitations

Although Bažant [4,7] claims that the endochronic theory of inelasticity in concrete is a good indicator of the response of plain concrete under multiaxial stresses--including strain-softening and failure envelopes, torsion-compression tests, lateral stresses, volume change, unloading and reloading diagrams and cyclic loading--except for close agreement between the theoretical predictions of the hysteresis with test data of three reinforced concrete beams, no other successful applications of the theory have been reported. There is a need to investigate ways of applying this theory to predict the hysteretic behavior of reinforced concrete critical regions which can be subjected to complex triaxial states of stress.

Biaxial stress-strain relationships--Darwin and Pecknold [6] formulated a nonlinear constitutive model for plain concrete subjected to cyclic biaxial stresses and applied it to the analytical prediction of the behavior of shear panels. The proposed constitutive model was based on the use of stress-strain curves similar to the uniaxial nonlinear stress-strain curve for plain concrete, and was claimed to be a significant improvement over elasto-plastic idealizations. However, Agrawal and Mufti [6] believe that the good agreement obtained by Darwin and Pecknold was due to allowing for two sets of open cracks and that good agreement can also be obtained using an elasto-plastic idealization. While Darwin and Pecknold state that bond slippage is not a major factor in deep shear walls, Shipman and Gerstle [6] show that even for the deep shear walls tested, the effects of bond slippage cannot be neglected.

From the above discussion, it is clear that no reliable constitutive model has yet been formulated for the case of biaxial stress states of reinforced concrete members, which is of utmost importance for the prediction of behavior of shear panels in structural wall systems.

RESEARCH AND DEVELOPMENT NEEDS

As discussed above, there are several major areas of knowledge regarding properties and behavior of currently used concrete which need further development or improvement before this composite material can be used effectively in combination with reinforcing steel to construct earthquake-resistant reinforced concrete buildings. Identification of these areas should be done taking into consideration that although the main concern is to improve knowledge of the behavior of concrete when subjected to the effects of all possible seismic ground motions likely to occur during a building's service life, these effects cannot be isolated from those induced by normal excitations and from the cumulative damage that can be induced by a series of consequential or independent abnormal events. Several research and development needs are described below.

1. To Improve Knowledge of the Mechanical Characteristics of Plain Concrete According to Properties of its Constituents

Plain concrete is a composite material made up of different types of aggregates, cement, water, and additives (admixtures). Usually, only the cylindrical compressive strength, f'_c, is specified and determined by testing. In most of the constitutive models that have been formulated, f'_c is the only parameter to be specified for a given concrete [7]. Alone, however, this mechanical characteristic is insufficient for predicting the performance of plain or confined types of concrete under seismic excitations. The following three parameters should also be considered.

(a) Modulus of elasticity--Although this parameter depends on strength and unit weight it is also very sensitive to the characteristics of the aggregate used.

(b) Deformability--To predict behavior of both the cover as well as that of the confined core of reinforced concrete elements under severe seismic excitations, it is necessary to know the complete stress-strain relationship of plain concrete, and not just up to the maximum strength.

(c) Poisson ratio--This parameter is needed to predict behavior of confined concrete.

2. To Improve Some of the Main Characteristics of Current Concrete Mixes

To attain more efficient earthquake-resistant concrete material, it will be necessary:

(a) to improve deformability of concrete, particularly in tension, and
(b) to increase ratios of both strength per unit weight and stiffness (modulus of elasticity) per unit weight.

3. To Determine the Effects of Aging and Environmental Conditions on Mechanical Characteristics of Concrete

Concrete is a building construction material very sensitive to the influence of the service history of the building. Very little is known about the effect of aging on the mechanical characteristics of concrete, particularly of concrete that is used in precasting, where different additives are used to obtain high-strength concrete in one-day steam-curing operations. The same is true for the mechanism of cumulative deterioration of concrete properties when subjected to severe changes in environmental conditions. Further study in this area is needed in order to predict damageability [27]. To predict seismic building response and potential damage it is necessary to assess the current state of the building at the time of the earthquake.

4. Development of New Types of Concrete

Normal-weight aggregate concrete has too low a strength per unit weight ratio for earthquake-resistant construction. The use of current lightweight aggregate concrete is attractive but there is a need to improve some of its properties or to develop new concrete mixes according to the needs stated under item (2).

5. To Determine Ways of Achieving Confined Concrete in Real Reinforced Concrete Structures, to Improve Knowledge of its Mechanical Behavior (particularly, Hysteretic Behavior under Seismic Excitations), and to Formulate more Reliable Constitutive Models

In reinforced concrete practice, confinement is usually achieved through properly designed and detailed lateral reinforcement. However, due to the need to protect the reinforcing steel against fires and corrosive environmental conditions, the confined core is covered by a shell of plain concrete. To obtain a full understanding of the behavior of the actual structural element, it is necessary to:

(a) study the behavior of the cover of confined concrete. Although the cover is of plain concrete, its behavior is affected by the presence and behavior of the confining lateral reinforcement;

(b) study the effect of spalling of the cover on the reduction of the effective resistant area of the confined concrete core (arch formation); and

(c) carry out comprehensive, integrated experimental and analytical studies of behavior of concrete confined by different arrangements of the confining lateral reinforcement. These studies should be conducted considering both externally applied uniaxial and multiaxial compressive states of stress under seismic conditions as well as under quasi-static increasing deformations.

6. To Establish Proper Test Specimens, Test Conditions and Instrumentation to Measure Deformations and Effective Resistant Areas of Confined Concrete and Proper Methods for Computing Stress-Strain Relationships of Confined Concrete

No two researchers use the same testing conditions, instrumentation or ways of computing the stress-strain relationship. Without standard procedures and methods, analyses of individual results are complicated and direct comparisons are usually invalid.

7. To Determine the Behavior of Stress Transfer (Bond) between Steel and Unconfined and Confined Concrete under Seismic Conditions

Reinforced concrete is a composite material of steel and concrete. Under severe seismic excitations, deformations concentrate in cracked regions. Hysteretic behavior of these cracked regions and their opening, closing, reopening, etc. are highly dependent on the interaction between steel and concrete (bond-slip relationship). There is presently little reliable data on which to base the formulation of a reliable bond-slip law.

8. To Improve Present Bond Characteristics and to Develop Methods of Bond Repair

The seismic design requirements for confinement and reinforcing of structural concrete members have been upgraded to such a degree that the triggering failure mechanism in such elements will be associated with the lack of adequate bond in the embedment length of continuous bars at joints or in the anchorage of the discontinuous bar, particularly in lapped splices. Bond deterioration in well confined concrete is due to internal cracking of the concrete and/or internal local crushing or shear of the concrete between the ribs of a deformed bar. Therefore, detection and repair of bond deterioration cannot be effected from the external surface of the members.

9. To Improve Knowledge of the Interaction between Flexure, Shear, and Axial Forces in Critical (Cracked) Regions of Reinforced Concrete Elements

Hysteretic behavior of reinforced concrete structures under severe earthquake excitations is controlled by the inelastic deformations induced by the combined effected of flexure, shear, and axial forces. These inelastic deformations are concentrated at the critical (cracked) regions. Thus, their prediction requires full understanding of the basic mechanisms of crack formation, propagation, and shear transfer along the cracks as affected by aggregate interlocking, friction, tie resistance, and dowel action. Integrated experimental and analytical studies should be conducted to gain such understanding before reliable constitutive models are to be formulated.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

STRENGTH AND DUCTILITY OF REINFORCED CONCRETE COLUMNS WITH RECTANGULAR TIES

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INTRODUCTION

This is a brief report on the results of a current research program at the University of Toronto. Examination of the state-of-the-art papers and recommendations submitted to the workshop indicate a need for information on the effectiveness of rectangular ties in columns. The Toronto project is at present about 60% complete. However, it appears that a preliminary reporting of the results obtained to date may be useful.

While most of the researchers agree that the rectangular ties improve the ductility of the confined concrete, there is a considerable difference of opinion about the increase in the load carrying capacity of the confined concrete core[1]-[23]. Factors causing the enhanced load carrying capacity and the ductility of the confined concrete core include the amount of lateral ties, distribution of column steel around the perimeter of the core, configuration of the tie steel and the relationship between the size of column steel and size and spacing of the lateral steel that delays the buckling of the column steel.

TEST PROGRAM

To date 18 square specimens with cross section of 12" x 12" (305m x 305mm) and 6'-5" (1960mm) long have been tested under monotonic axial compression to failure. Ends of specimens were enlarged and confined with steel plates to prevent premature end failures. Columns were cast vertically, in steel forms, with six specimens being cast at the same time. In all specimens dimension of the core (measured from center to center of perimeter hoop) was 10.5"x10.5" (267 x 267mm). Dimensions of the specimens are given in Figure 1.

TEST VARIABLES

Test variables for each speciman are given in Table I. Following is a brief summary of the variables examined or planned for examination.

1. Distribution of Column Steel Around Perimeter

There are no reasons why well distributed longitudinal column steel should not improve the confinement mechanism of the concrete core. Most of previous tests have been performed on specimens with four corner bars. To investigate the significance of this effect, columns are constructed with 8 or 12 or 16 uniformly distributed longitudinal bars. The resulting cross sections and tie configurations are shown in Figure 1. So far only the configuration A and C have been investigated.

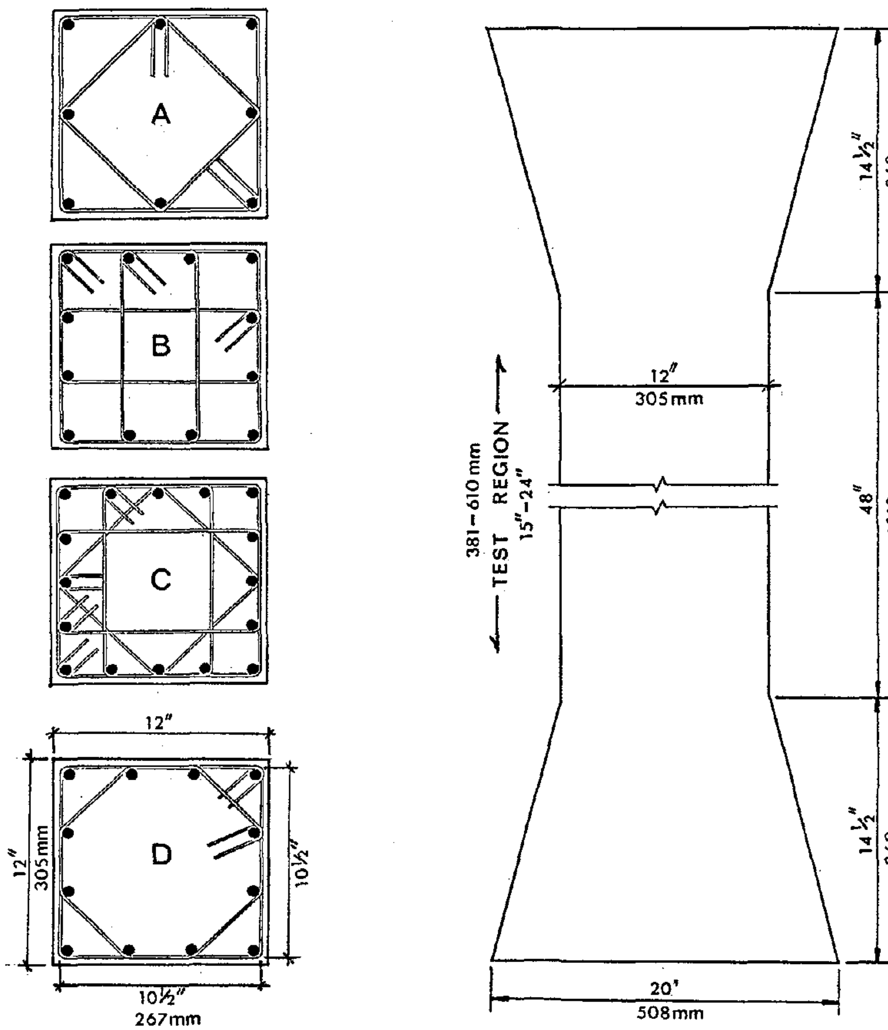

Figure 1: Detailing of Test Specimens

TABLE I: AXIALLY LOADED SHORT COLUMNS

12" x 12" x 6'-5"; Core: 10.5" x 10.5" (to ℄ of hoop)

SPECIMEN	COLUMN STEEL					LATERAL STEEL			CONC.	COMPUTED			EXPERIMENTAL			
	No. & Size	A_s (in)	f_y (ksi)	ρ(%) Gross	ρ(%) Core	d_h (in)	s (in)	ρ_s(%)	f'_c (ksi)	P_o (kips)	P_{oc} (kips)	P_{occ} (kips)	P_{test} (kips)	P_{cmax} (kips)	ϵ_{cmax}	$\frac{P_{cmax}}{P_{occ}}$
2A1 - 1	8 #5	2.48	54.00	1.72	2.25	3/16	2.25	0.80	5.44	788	654	498	768	636	0.0038	1.28
2A1H- 2	8 #5	2.48	54.00	1.72	2.25	3/16	2.25	0.80	5.37	779	645	491	755	624	0.0057	1.27
4C1 - 3	16 #5	4.96	54.00	3.44	4.50	1/8	2.00	0.76	5.28	891	624	472	850	586	0.0039	1.24
4C1H- 4	16 #5	4.96	54.00	3.44	4.50	1/8	2.00	0.76	5.32	896	628	476	860	593	0.0024	1.25
4C6 - 5	16 #5	4.96	54.00	3.44	4.50	3/16	1.50	2.27	5.07	867	599	453	1058	745	0.0150	1.65
4C6H- 6	16 #5	4.96	54.00	3.44	4.50	3/16	1.50	2.27	4.98	856	588	445	950	685	0.0066	1.54
4A3 - 7	8 #7	4.80	56.00	3.33	4.35	5/16	3.00	1.66	5.93	970	701	531	960	692	0.0034	1.30
4A4 - 8	8 #7	4.80	56.00	3.33	4.35	3/16	1.13	1.59	5.92	969	700	530	992	724	0.0052	1.37
4A5 - 9	8 #7	4.80	56.00	3.33	4.35	3/8	3.00	2.39	5.88	964	695	527	921	650	0.0045	1.23
4A6 - 10	8 #7	4.80	56.00	3.33	4.35	1/4	1.38	2.32	5.90	966	698	528	975	700	0.0099	1.33
4C3 - 11	16 #5	4.96	59.00	3.44	4.50	1/4	3.75	1.62	5.90	989	697	528	958	668	0.0056	1.27
4C4 - 12	16 #5	4.96	59.00	3.44	4.50	1/8	1.00	1.52	5.92	992	699	529	1105	772	0.0073	1.46
4A1 - 13	8 #7	4.80	63.50	3.33	4.35	3/16	2.25	0.80	4.54	841	537	406	852	546	0.0038	1.34
2A5 - 14	8 #5	2.48	58.50	1.72	2.25	3/8	3.00	2.39	4.57	694	549	418	725	578	0.0050	1.38
2A6 - 15	8 #5	2.48	58.50	1.72	2.25	1/4	1.38	2.32	4.60	698	553	421	780	622	0.0057	1.48
2C1 - 16	16 #4	3.20	60.00	2.22	2.90	1/8	2.00	0.76	4.72	756	564	429	779	590	0.0060	1.38
2C5 - 17	16 #4	3.20	60.00	2.22	2.90	5/16	4.00	2.37	4.77	762	570	434	792	593	0.0070	1.37
2C6 - 18	16 #4	3.20	60.00	2.22	2.90	3/16	1.50	2.27	4.80	766	574	436	1004	744	0.0253	1.71

2. Amount of Lateral Reinforcement

Volumetric ratio of tie steel (ρ_s) was varied. Values used were approximately 0.8 or 1.6 or 2.4% of the volume of core concrete.

3. Tie Spacing(s)

The ratio of the tie spacing to the dimensions of the confined core can be an important parameter in determining the column behaviour. Smaller spacing should lead to more effective confinement, and small spacing of ties may also delay the buckling of the longitudinal bars.

A number of pairs of columns have been tested with tie spacing as a variable while keeping the volumetric ratio of transverse steel constant.

4. Amount of Longitudinal Reinforcement

The ratio of the area of longitudinal reinforcement to the gross section area was varied between 0.017 and 0.034. This variable was investigated by comparing columns of otherwise identical nature.

5. Characteristics of Transverse Steel

The effect of yield strength and stress-strain characteristics of the tie steel on the confinemement of the core is examined. Lateral steel was heat-treated in some columns for the purpose of re-introducing a flat yield plateau.

EXPERIMENTAL WORK AND INSTRUMENTATION

The alphanumeric characters in the titles of the columns (e.g. 4C6H-6) indicate the following:

- The first number represents the amount of longitudinal reinforcement rounded off to a whole number percentage (2% or 4%). The letter after the first number indicates the configuration of reinforcement in the column section.

- The second number refers to the amount of lateral reinforcement. The letter 'H' if present after the second number indicates that the lateral reinforcement has been heat treated.

- The last number is the sequence number of the specimens tested and is used as a convenient reference.

To ensure that failure occurs in the instrumented portion of the column the spacing of the sets of ties outside the test region was reduced. In the first set of 6 specimens the length of the test region varied between 15 - 18 inches (381-457mm) while in the other 12 specimens it was about 24". In all tests failure occurred in the instrumented region.

Test data included measurements of strains in both the longitudinal bars and the stirrup reinforcement. Longitudinal and lateral strains in concrete were also measured on all four sides of the column using a Pfender gage over a gage length of 100mm (4in). Eight bars in all columns were instrumented, four middle bars with electric strain gages and four corner bars with dial indicators. In the first 6 columns the gage length for the dial indicators was 12 inches (305mm) and in all other columns it was 14 inches (356mm). Each stirrup in one set of stirrups in all the columns were instrumented with electric strain gages.

Two LVDT's one on the west and the other on the east side of the column, were installed to obtain the load deformation behaviour of the test region. Movement of the machine head was also plotted against the load for all columns except column 4C6H-6. In the first 6 columns the gage length for the LVDT measurements was 19 inches (483mm) and in all other specimens it was 21 inches (533mm).

All columns were loaded in monotonic, concentric compression in the 1200 kips Baldwin Universal testing machine. While strain readings were taken, the load was kept constant. The total testing time from the start of loading to the end of the test varied from 3 to 6 hours for each specimen.

RESULTS

The preliminary results reported here are obtained by the use of the LVDT's in the form of load vs. Average strain in the column. The contribution of concrete in carrying the load was calculated by subtracting the steel contribution from the total applied load at a particular strain level as shown in Figure 2 for column 4C6-5. In doing so, it is assumed that the strain in concrete is equal to strain in longitudinal steel. A comparison of strain in the longitudinal steel and the vertical column strain shows that this assumption is quite valid for the strain range considered in the tests.

Definition of terms

A_g = Gross Area ($144.0 in^2$)

A_c = Core Area (measured to centre of perimeter hoop, $110.3\ in^2$)

f_s = Steel stress at the strain level considered

P_{test} = Total axial force applied to the column

P_{st} = Force resisted by the column steel ($A_s f_s$)

$P_{conc} = P_{test} - P_{st}$

$P_{oc} = 0.85\ f'_c\ (A_g - A_s)$

$P_{occ} = 0.85\ f'_c\ (A_c - A_s)$

$P_o = P_{oc} + A_s f_y$

P_{cmax} = Maximum test load carried by the concrete: ($P_{test} - P_{st}$)

ε_{cmax} = Average longitudinal strain at P_{cmax}

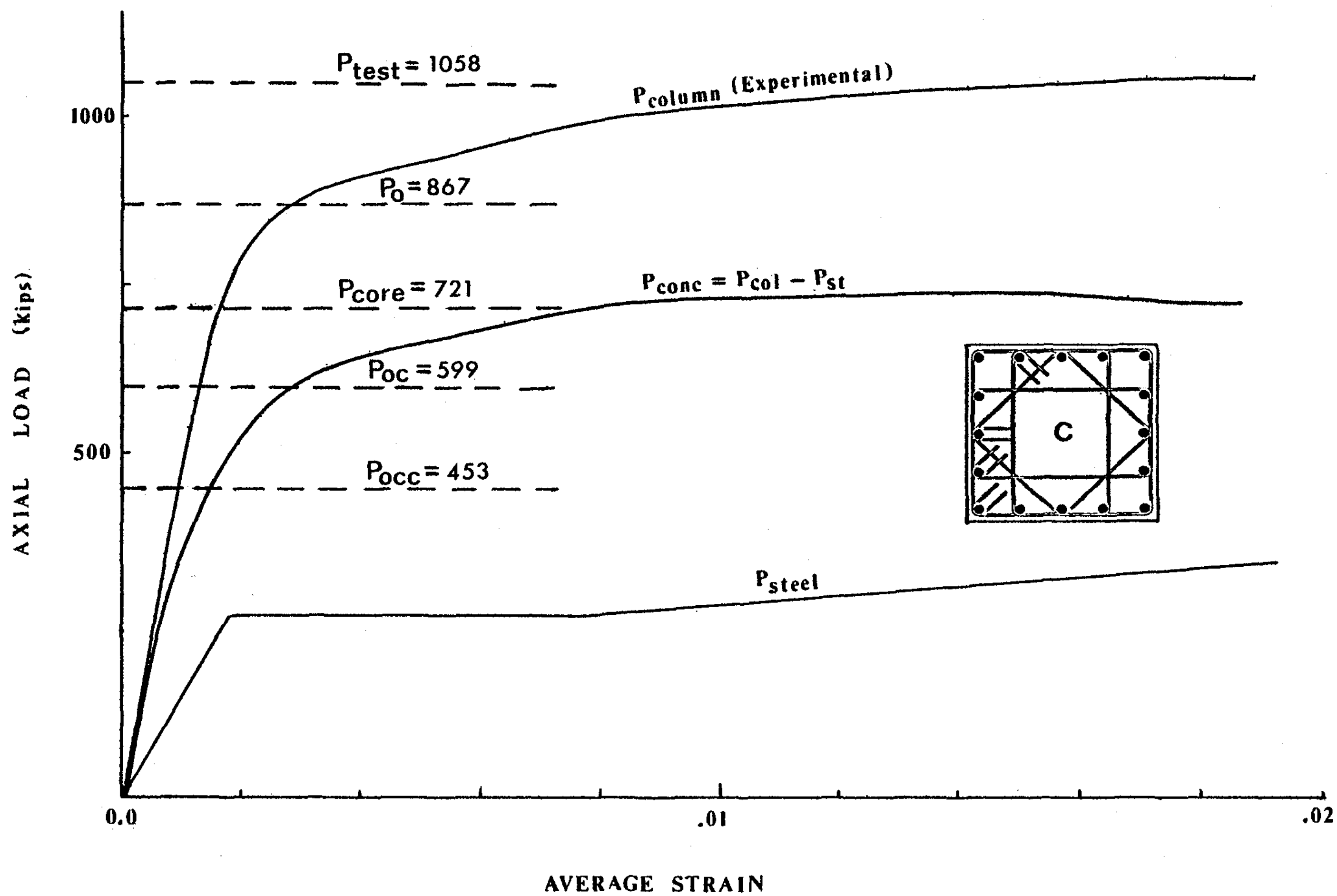

Figure 2: Calculation of Concrete Contribution from the Total Load for column 4C6-5

Concrete Contribution

To facilitate a comparison of the behaviour of different columns the concrete contribution was non-dimensionalized with respect to the total concrete force (P_{oc}) and the core concrete force (P_{occ}). Thus there are two graphs for each column showing concrete contribution. The lower one was obtained by dividing the concrete load by P_{oc} and the upper one resulted by dividing the concrete force by P_{occ} as shown in Figure 3.

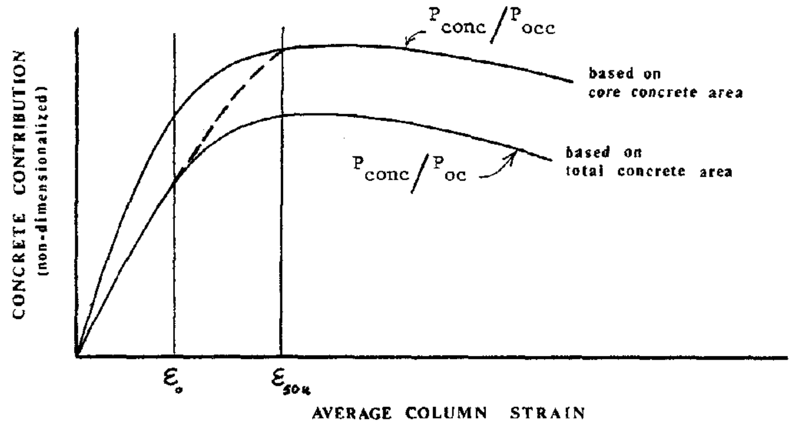

Figure 3: Concrete Contribution curves normalized with respect to total concrete area and core concrete area.

Results of cylinder tests show that at a strain close to 0.002 (ε_o) the stiffness of concrete is very close to zero, and the maximum stress occurs at approximately this strain. If it is assumed that the concrete in the cover of the column behaves in a way similar to the concrete in the cylinder, then the cover should start spalling off at about 0.2% strain (ε_o). This is the approximate point at which the lower curve in Figure 3 ceases to represent the behaviour of column concrete. Assuming again that cover concrete is not effective in load carrying beyond a strain of .004 - .005 (ε_{50u} - strain at which the stress drops to 50% of ultimate in concrete cylinder), the upper curve will represent the concrete behaviour beyond this strain. Between the strain values of ε_o and ε_{50u} a transition takes place from the lower curve to the upper curve.

In this preliminary report no attempt is made to define the shape and exact ends of the transition curve. Maximum concrete force (P_{cmax}) in the column obtained from the tests and the corresponding maximum strains (ε_{cmax}) are listed in Table I. The use of the value of (ε_{cmax}), without referring to the load deformation curves would be misleading in many cases. In addition the ratio of the maximum test load carried by the concrete (P_{cmax})

to the unconfined strength of the core concrete (P_{occ}) are given in Table I.

Some of the results are also given in Figures 4 - 9. These figures show the variation of (P_{conc}/P_{oc}) or (P_{conc}/P_{occ}) with the average column strain The relevant portions of these curves are the portions after the spalling of the cover which indicate the increased load carrying capacity of the confined concrete core.

Figures 4 to 9 show the effect of tie configuration (Figures 4 and 5); the amount of lateral reinforcement (Figures 6 and 7) and the effect of tie spacing (Figures 8 and 9) on the additional load carrying capability and the ductility of the confined concrete core.

CONCLUSIONS

On the basis of the preliminary examination of the tests performed so far, it may be concluded that:

1. Concrete when confined with rectangular ties and longitudinal steel exhibit a very significant strength gain as well as increased ductility.

2. Good distribution of the main column steel around the perimeter enhances the strength of the confined core (Figures 4 and 5).

3. Amount of lateral reinforcement has a very significant effect on the strength of the confined core (Figures 6 and 7).

4. Increased spacing of the rectangular ties, even with the same volumetric ratio of lateral steel, results in reduction in the strength gain of the confined core (Figures 8 and 9).

ACKNOWLEDGEMENTS

Research reported is supported by a National Research Council of Canada grant (NRC A4273). Experimental work was conducted in the Structures Laboratory of the Department of Civil Engineering of the University of Toronto.

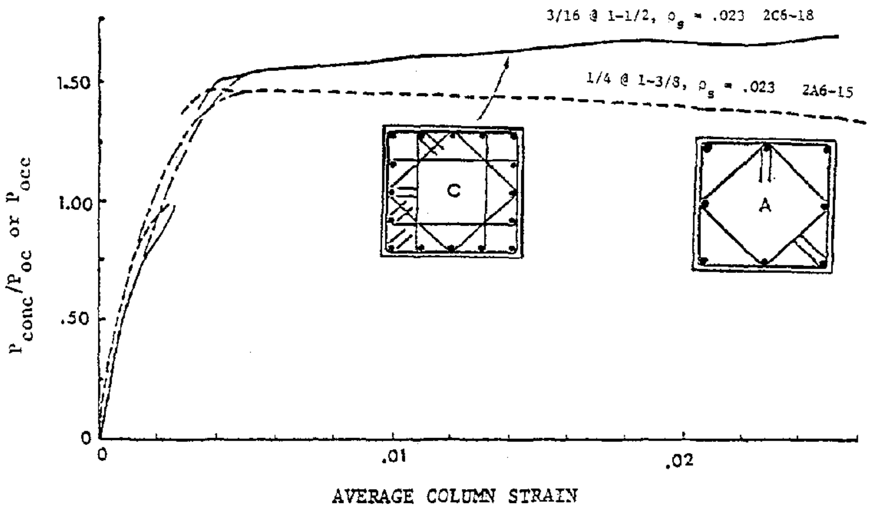

Figure 4. Effect of Tie Configuration

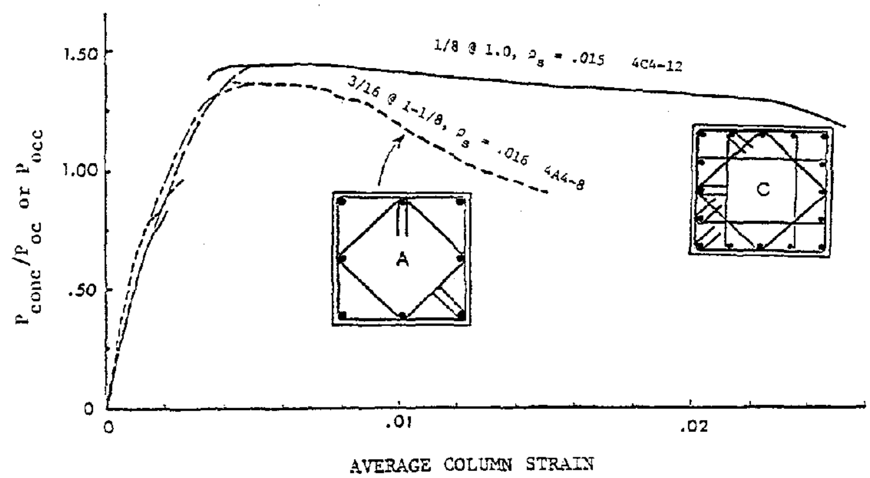

Figure 5. Effect of Tie Configuration

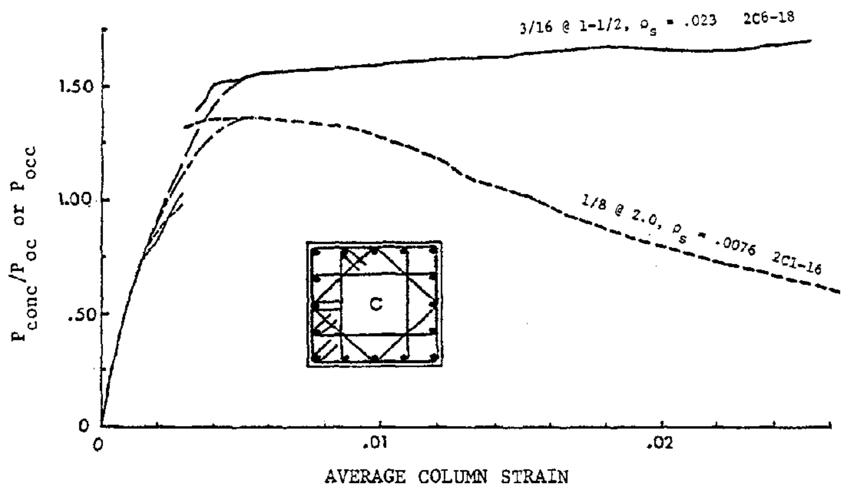

Figure 6: Effect of Amount of Lateral Reinforcement

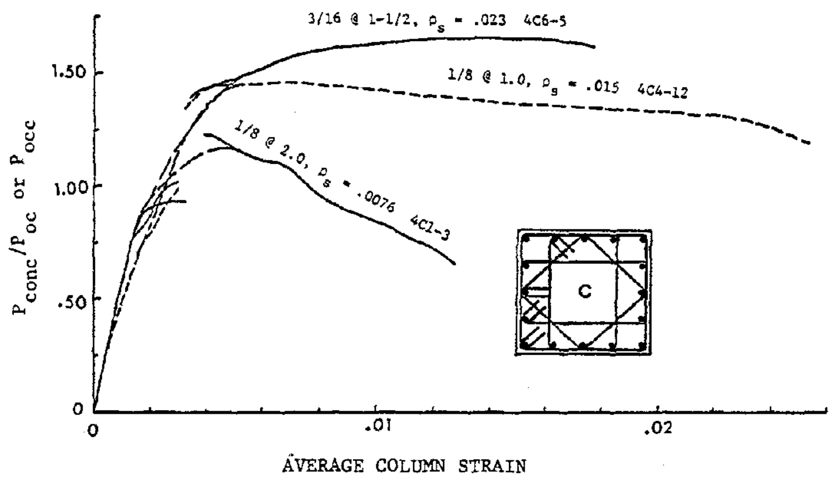

Figure 7: Effect of Amount of Lateral Reinforcement

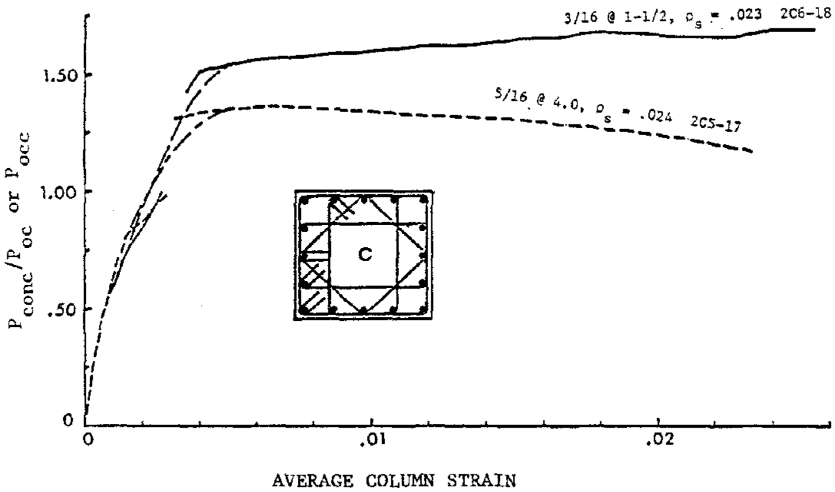

Figure 8: Effect of Tie Spacing

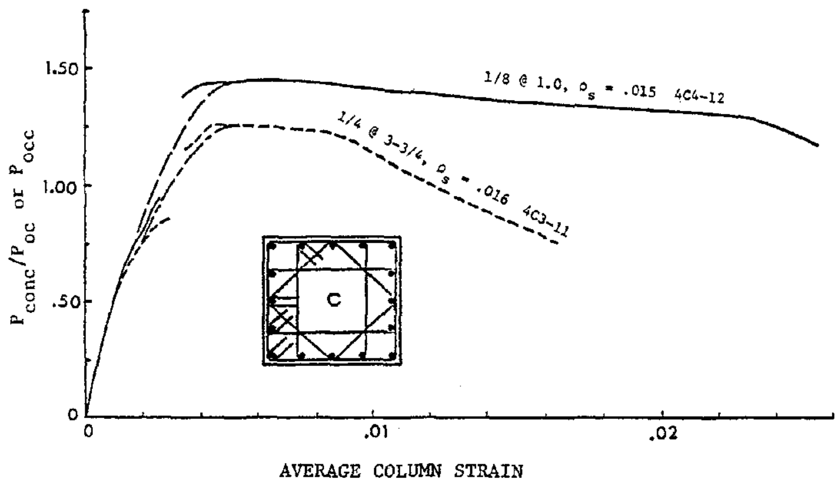

Figure 9: Effect of Tie Spacing

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED
CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

A NOTE ON THE FAILURE CRITERION FOR DIAGONALLY CRACKED CONCRETE

by

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INTRODUCTION

It is generally agreed that it is necessary to eliminate the possibility of shear failures if the ductile response of reinforced concrete structures is to be ensured. When checking the shear capacity of diagonally cracked elements such as shear walls, beams, columns and joints it is often appropriate to assume that all of the shear is resisted by diagonal compressive stresses in the cracked concrete (i.e. by a compression field) [1,2]. The difficulty then is to decide what is the appropriate limiting value of the principal compressive stress in the concrete. It will be appreciated that because the stress must be transmitted across cracked and possibly severely deformed concrete it is unlikely that the diagonal compressive stress can reach the cylinder crushing strength (f'_c) of the concrete.

After briefly describing a failure criterion that has been proposed for monotonically loaded diagonally cracked concrete this note will discuss the possibility of applying a similar criterion for diagonally cracked concrete subjected to reversed cyclic loading.

CRITERION FOR MONOTONICALLY LOADED CONCRETE

For monotonically loaded diagonally cracked concrete it has been proposed [2] that the stress conditions that exist in the concrete at failure are related to the strain conditions in the concrete at failure.

The stress conditions in the diagonally cracked concrete can be represented by a Mohr's circle of stress such as that shown in Fig. 1. It can be seen from Fig. 1 that if it's desired to avoid tensile stresses in a concrete element subjected to shear stresses on "longitudinal" and "transverse" planes then compressive stresses must exist in both the "longitudinal" and "transverse" directions. These compressive stresses must be equilibrated either by external axial loads or by tensile stresses in the reinforcement.

For the case of no tensile stresses in the concrete it is possible to derive (from Fig. 1) the following relationship between the applied shear stress, v, and the resulting maximum diagonal compressive stress, f_d

$$f_d = v \tan \alpha + v/\tan \alpha \qquad \ldots (1)$$

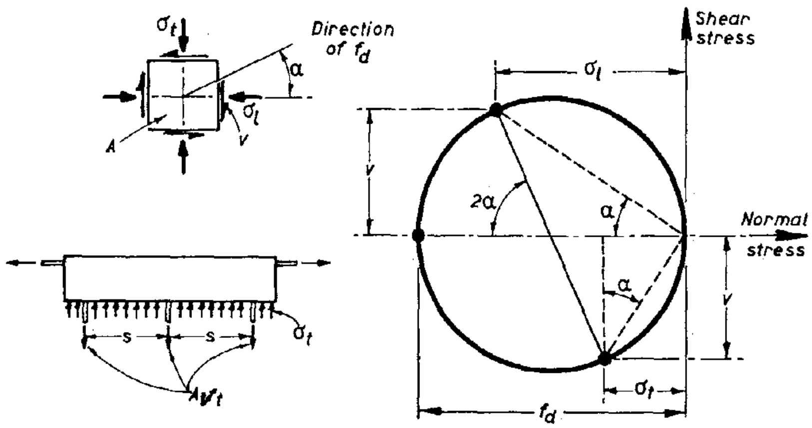

Figure 1. Equilibrium Conditions for the Average Stresses in the Concrete

where α is the angle between the longitudinal axis and the direction of the maximum diagonal compressive stress. The angle α is a function of the ratio of the magnitude of the longitudinal compressive stress, σ_ℓ, to the magnitude of the transverse compressive stress, σ_t.

The strain conditions in the diagonally cracked concrete can be represented by a Mohr's circle of strain such as that shown in Fig. 2. This circle

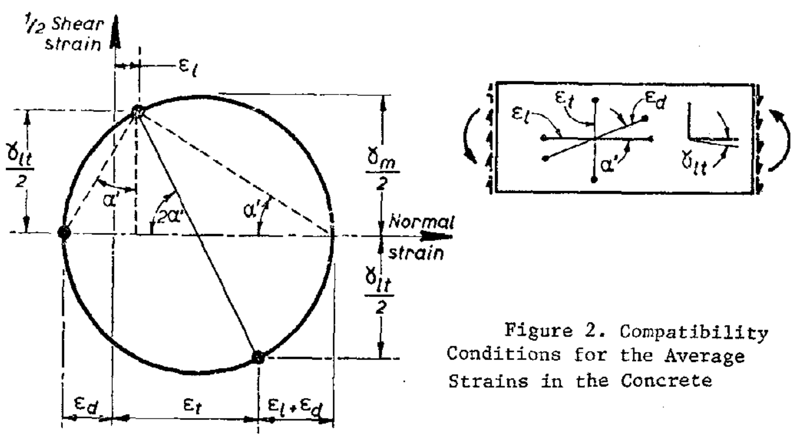

Figure 2. Compatibility Conditions for the Average Strains in the Concrete

depicts the relationships which must exist between the average strains (i.e. the strains "measured" over a base length which is several times the crack spacing) in the various directions.

In constructing Fig. 2 it has been assumed that there is a large tensile strain, ε_t, in the transverse direction, a smaller tensile strain, ε_ℓ, in the longitudinal direction and a maximum compressive strain of ε_d.

It can be seen from Fig. 2 that the maximum shear strain in the concrete γ_m (i.e. the diameter of the strain circle) is given by

$$\gamma_m = \varepsilon_t + \varepsilon_\ell + 2\varepsilon_d \qquad \text{... (2)}$$

or by

$$\gamma_m = \gamma_{\ell t} \,/\, \sin 2\alpha' \qquad \text{... (3)}$$

where α' is the angle of inclination of the principal compressive strain.

It has been proposed [2] that the size of the stress circle, Fig. 1, that causes the concrete to fail is related to the size of the co-existing strain circle, Fig. 2. The diameter of the strain circle, γ_m, was suggested as the indicator of strain intensity while the diameter of the stress circle at ultimate, f_{du}, was taken as the indicator of stress intensity.

On the basis of a number of experimental results for monotonically loaded beams subjected to "pure shear" it was suggested [2] that a reasonable estimate of the value of f_{du} would be given by:

$$f_{du} \,/\, f'_c = 3.6/(1 + 2\gamma_m/\varepsilon_o) \qquad \text{... (4)}$$

DIAGONALLY CRACKED CONCRETE SUBJECTED TO REVERSED CYCLIC LOADING

It should be appreciated that the failure criterion described by Equation 4 does not predict exactly how the diagonally cracked concrete will fail. It merely states that for a particular average strain condition in the concrete there will be a maximum size of stress circle that can be resisted by the concrete. Physically the concrete may for example fail on one critical plane because the shear stress exceeds the maximum that can be transmitted for the particular combination of axial stresses and crack widths which exist on this plane.

Because the proposed criterion uses average strain as an indicator of deformation and concrete cracking the criterion will probably not be appropriate when the deformation concentrates at one critical crack location as is the case of "sliding shear" failures [1]. However, when the crack growth and resulting deformation is relatively uniformly distributed over a member it is possible that a criterion similar to Equation 4 may be appropriate even for the case of reversed cyclic loading. The reversed cyclic loading will of course "soften" the concrete and lead to larger deformations (i.e. larger values of γ_m). A criterion such as that proposed would predict that this increase in deformation will lower the value of diagonal compressive stress required to cause failure.

The recent shear wall tests at PCA [3] provide some of the data required to investigate the validity of the criterion for reversed cyclic loading. Fig. 3 compares the experimentally determined values of f_{du} with the experimentally determined values of γ_m for a number of specimens. Five of the PCA walls which failed by "web crushing" have been plotted (B2, B5, B5R, B7 and F1) along with the results of four monotonically loaded beams (CF1, SA2, SA3 and SA4). The point for the cylinder was plotted on the basis of a Poisson's ratio at ultimate for the cylinder of 0.3. On the basis of these results it seems possible that a failure criterion for diagonally cracked concrete subjected to reversed cyclic loading may not be significantly different from that derived for the monotonically loaded beams (i.e. Equation 4).

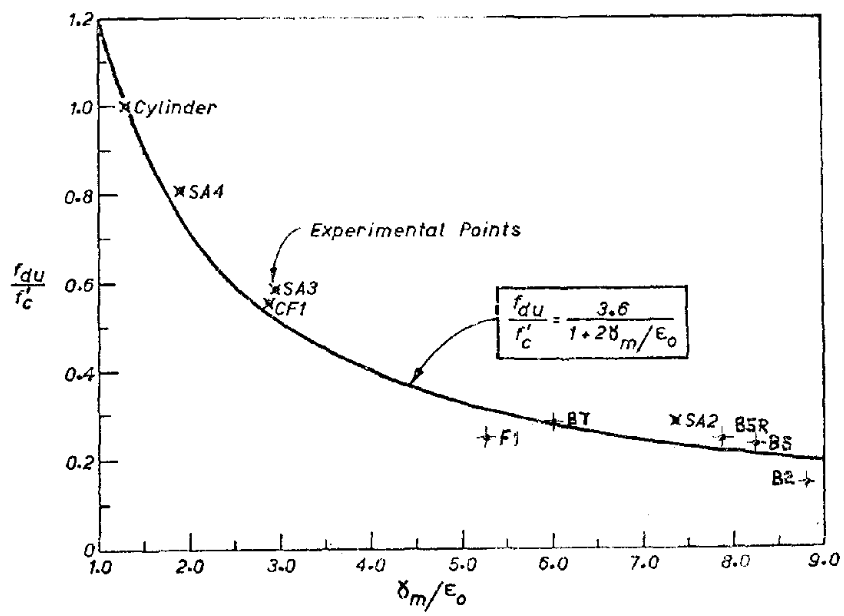

Figure 3. Principal Compressive Stress at Failure as a Function of Shear Strain

CONCLUDING REMARKS

It should be stressed that even if a failure criterion such as that proposed proves to be adequate for some cases of reversed cyclic loading a significant body of information is still required before the criterion can be used to predict shear capacity. In particular we need to be able to predict for any given load history the value of γ_m. In order to do this we need information on the stress-strain characteristics of diagonally cracked concrete. For monotonically loaded diagonally cracked concrete some information is available [2] on the relationships between principle compressive stresses and principle compressive strains. However, essentially nothing is available in the literature on the presumably degrading stiffness properties of concrete diagonally cracked in both directions and subjected to reversed, cyclic loading.

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MECHANICAL CHARACTERISTICS AND PERFORMANCE OF REINFORCING STEEL UNDER SEISMIC CONDITIONS

by

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INTRODUCTION

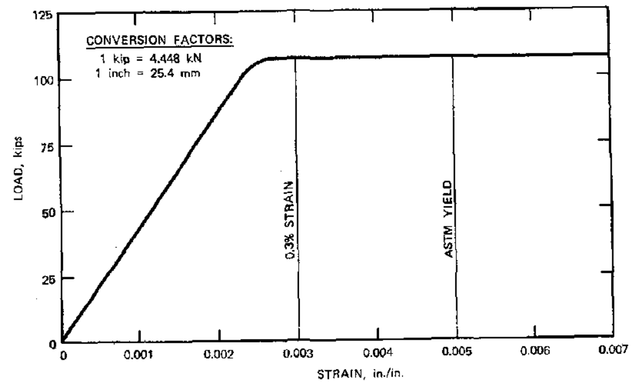
During the past decade, greatly increased emphasis has been placed on evaluating the suitability of different reinforcing steels for concrete structures subject to earthquakes. Various opinions on this matter have been expressed on the basis of past earthquake experience, tests of reinforced-concrete structures strained inelastically, and theoretical structural analyses. Discussions, both written and oral, have concerned the mechanical properties that reinforcing bars and prestressing steel should have to provide optimum earthquake resistance for concrete structures. This has led to deliberations by code-writing committees, technical committees of trade organizations, and other research-sponsoring agencies. As a result of this activity and other incentives, some research has been conducted that relates, either directly or indirectly, to the performance of reinforcing steel in concrete structures subject to seismic motions. Finally, the net effect of these various efforts tempered by practical limitations encountered in the production of reinforcing steels has culminated in material and design specifications for structures constructed in seismic zones.

This paper first traces these recent developments as related to the seismic performance of reinforcing steel. Included is pertinent research that was not specifically intended to solve seismic problems. Theoretical analyses were then made of what the author considers to be logical approaches in identifying quantitatively the reinforcing-steel parameters pertinent to seismic performance. These analyses result in a series of propositions that are intended to be a basis for Workshop discussions. Debatable points in these propositions and questions not answered by the propositions could form the basis of suggestions for future research.

RECENT DEVELOPMENTS

Significant Properties of Reinforcing Bars

Yielding and strain hardening—The mechanical properties of reinforcing steel most pertinent to seismic performance relate to the yield and post-yield behavior, as described by a tensile engineering stress-strain curve for the material. For reinforcing bars in nonprestressed flexural members, Figure 1, a well-defined yield point followed by an extensive level yield



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PLATEAU-YIELDING LOAD-STRAIN CURVE FOR A NO. 11 A615 GRADE 60 STEEL REINFORCING BAR

Source: U.S. Steel Corporation — Figure 1

plateau[1]* has been considered preferable, so that the tensile steel can be designed to yield at a specific stress and allow considerable inelastic rotation within a region of plastic hinging without local crushing of the concrete on the opposite side of the member. (A numerical evaluation of this phenomenon is given later.) An upper limit on the as-furnished yield strength is desirable for that reason and also so that an unexpectedly high tensile-steel stress will not induce an excessive bending moment in the member. The main undesirable effect of a greater moment is the concommitant greater shear and, as a result, the possibility of a premature stirrup or diagonal tension failure. However, it is realized that strain hardening of the steel is generally necessary for causing some increase in the resisting moment so that the region of plastic hinging can be spread over a reasonable increment of the member length, Figure 2. Without some increase in resisting moment, the moment gradient that almost always occurs within a finite length of plastic hinging could not be accommodated, and all the inelastic stretching of the reinforcing steel would occur at one location, very likely resulting in steel rupture.

Ductility and energy absorption--Ductility has been considered to be a very important property of reinforcing bars subjected to seismic straining. However, there has not been general agreement on how ductility should be defined or just how much ductility is required. Ductility can be defined

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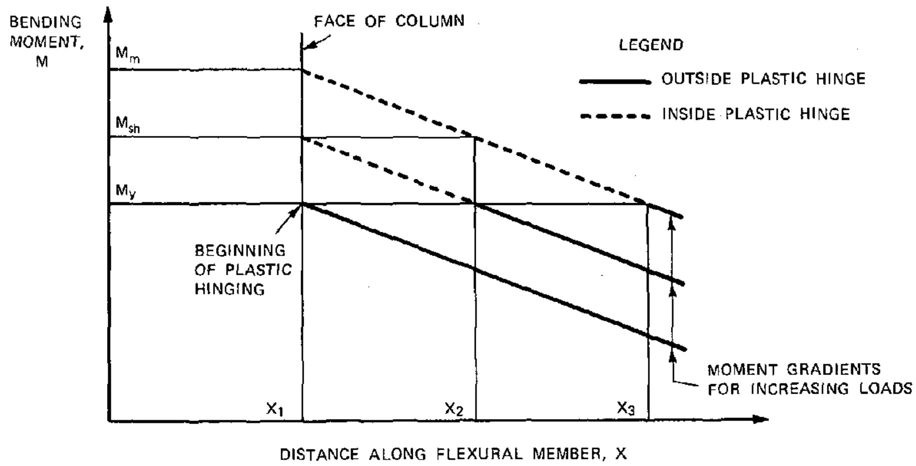
* See References.



M_y = MOMENT AT FIRST YIELDING OF TENSILE STEEL

M_{sh} = MOMENT AT ONSET OF STRAIN HARDENING OF TENSILE STEEL

MOMENT GRADIENT OVER REGION OF PLASTIC HINGING

Figure 2

alternatively (1) as the strain just before unloading, that is, the strain corresponding to the high point on the engineering stress-strain curve, (2) as the abscissa of the engineering stress-strain curve at rupture, or (3) computed in accordance with ASTM from the increase in the distance between the gage points straddling the rupture as measured when the two pieces of the broken test specimen are fitted together again. The first ductility has been termed the "useful ductility" because rupture is then eminent if the loading is not immediately decreased. This ductility is not nearly as sensitive to the gage length for measuring the strains as are the second and third ductilities. It has been frequently noted that ductility can be critical in the vicinity of a weld, particularly if the steel composition or the welding procedure is not appropriate.

Within the cyclic motion typical of earthquakes, the kinetic energy of the motion must be converted periodically into strain energy, both stored elastically and absorbed plastically. Because of its brittleness and low tensile strength, nonprestressed concrete cracks early in the strain history and is not very effective in absorbing energy before concrete crushing. Therefore, the reinforcing steel in nonprestressed concrete receives a major portion of the energy absorption. It has been noted[2] that optimizing the steel energy-absorption capacity involves using a low enough reinforcing index

$$\frac{(\rho - \rho') f_y}{f_c'}$$

so that the steel will yield considerably before concrete crushing on the other side of the member, but using a high enough reinforcing index so that the steel will not rupture before the concrete crushes; ρ and ρ' are the tension and compression steel percentages, respectively, f_y is the steel yield strength, and f_c' is the concrete compressive strength. With the reinforcement index set, steel energy-absorption capacity will be maximum if the ductility is not exceeded before the concrete crushes.

Concerns regarding bar properties—A615 reinforcing bars[3] with a specified yield point not exceeding 40 ksi (276 MPa) have been observed to have most of the attributes listed above, that is, a well-defined yield point, a yield plateau, adequate strain-hardening capacity, adequate ductility, and generally satisfactory reinforcing indices in typical designs. However, occasional doubts have been expressed regarding whether these characteristics can be consistently achieved in reinforcing bars with specified yield strengths exceeding 40 ksi. As discussed below, doubts about the seismic application of A615 reinforcing bars[3] with a specified minimum yield strength of 60 ksi (414 MPa) have mostly stemmed from a lack of knowledge of the current as-produced mechanical properties of grade 60 rebars and of the quantitative values of the yield strain, strain hardening, ductility, and other properties required for reinforcing bars in seismic structures.

For reinforcing bars of all specified strengths, there has been concern regarding the use of welded connections in structures that could be subject to seismic behavior. Both the strength and ductility of welded connections have been questioned because of the absence of restrictions in the bar chemical composition (except for a limitation on phosphorus) in the ASTM specifications before 1974.

Consideration of Welded Wire Fabric

As discussed herein, "reinforcing bars" refers to bars and rods with external deformations that are used in non-prestressed applications. The other reinforcement used extensively in concrete structures is welded wire fabric, fabricated with either smooth or deformed wires. In above-grade applications, welded wire fabric is used extensively as slab reinforcement and sometimes as peripheral cage or stirrup reinforcement in columns or beams. However, it is not generally used as the main longitudinal reinforcement in beams or columns. Therefore, relatively little information is available about the suitability of welded wire fabric for seismic applications. Because the ductility of welded wire fabric is generally significantly less than that of reinforcing bars, no major efforts have been made to evaluate the suitability of fabric in seismic structures.

Significant Properties of Prestressing Steel

As used in prestressed concrete, prestressing wires and strand do not generally exhibit all the mechanical properties considered desirable for seismic reinforcing bars. However, it has not been considered either necessary or desirable for prestressing steel to have the same seismic properties as rebars. The main reason is that the requirements for satisfactory plastic hinging are different for prestressed concrete. Because of the longitudinal prestress, the concrete undergoes considerable rotation with significant energy stored in the strained concrete before the concrete cracks and before the minimum specified ductility of the prestressing steel is exceeded. With the concrete in longitudinal compression, the problem of premature stirrup or diagonal tension failures resulting from higher shears caused by higher moments is much less critical than it is in nonprestressed concrete. In general, the mechanical properties of currently available prestressing steel have not been considered a problem in seismic applications.

Experimental and Theoretical Investigations and Evaluations

During the past decade, experimental and theoretical investigations have been made that have answered many of the questions regarding the suitability of different reinforcing steels in seismic structures. The major portion of this activity was concerned with reinforcing bars in nonprestressed concrete, and a limited portion involved prestressing steel.

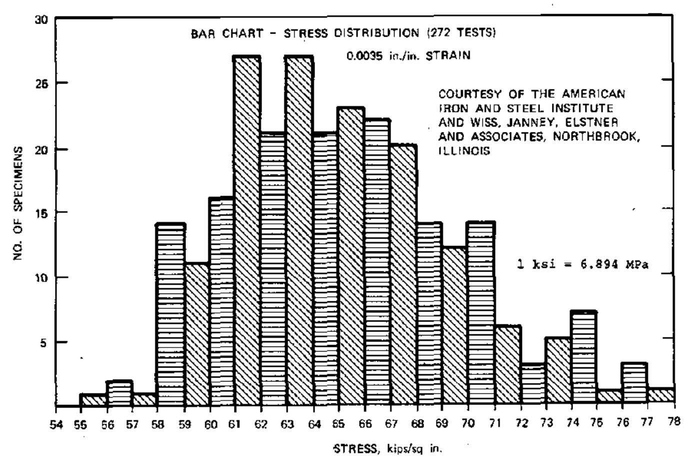
PCA—The Portland Cement Association (PCA) initiated their seismic experimental research with reverse-loading tests on full-scale cross-shaped specimens, which represented the column-girder joints of rigid frames. During the first test series,[4] the reinforcement was all grade 40 (40-ksi or 276-MPa specified minimum yield point). During the second test series,[5] which was jointly sponsored by PCA and the American Iron and Steel Institute (AISI), the main longitudinal reinforcement was all grade 60 (60-ksi or 414-MPa specified minimum yield strength) and the specimens were designed to have the same static strength as comparable specimens in the first test series. A comparison of the load-deflection curves of the two series indicated no difference between the performance of grade 40 and grade 60 steels under seismic loadings. Somewhat greater strain hardening was indicated for the grade 60 steel (23% average increase and 41% maximum increase in moment over the yield moment) than for the grade 40 steel (10% average increase and 25% maximum increase). Even with a deflection ductility ratio of 5.0 induced in each test specimen, the steel strains measured by attached strain gages did not exceed about 4 percent in any of the tests.

Subsequently, PCA conducted an extensive series of tests on shear walls. In these tests also, the maximum measured steel strains were low, generally not exceeding about 3 percent.[6] The only longitudinal steel ruptures were secondary failures, indicated by column buckling of the bars.

WJE—During the past decade, the steel industry has expressed concern that various code-writing bodies may be composing impractical specifications for reinforcing steel because of a lack of knowledge of the mechanical properties consistently exhibited by the steel. As a result, the steel industry sponsored three experimental projects conducted by the firm of Wiss, Janney, Elstner, and Associates (WJE). These projects are pertinent to the present discussion, even though they were not initiated as seismic investigations.

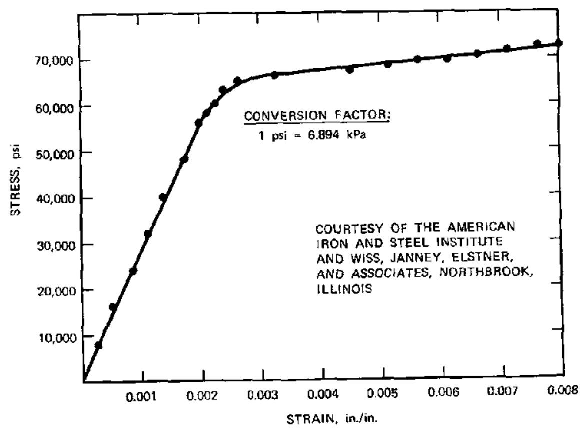
Tension tests by WJE for the American Iron and Steel Institute (AISI) on an industry-wide sampling of grade 60 reinforcing bars[7] indicated that grade 60 rebars have a well-defined yield point at a stress generally not exceeding 78 ksi (538 MPa), Figure 3, followed by a gently sloping yield "ramp," Figure 4, rather than by a flat yield plateau. Groupings of the stress-strain data points are given in the Appendix A figures for strains up to 0.8 percent and in the Appendix B figures for greater strains. Generally, stresses in excess of about 78 ksi did not occur until a strain of over 0.8 percent had been obtained. The minimum ductility specified by ASTM for grade 60 steel was generally obtained.

A similar series of tension tests by WJE for the Wire Reinforcement Institute (WRI) on an industry-wide sampling of smooth wire[8] indicated that wire for welded wire fabric typically exhibits a round-house stress-strain curve, and that the ductility (which is not specified by ASTM) typically does not exceed about 6 percent, and, occasionally, may be as low as 1



HISTOGRAM OF STRESS DISTRIBUTION FOR INDUSTRY-WIDE SELECTION OF GRADE 60 REINFORCING BARS

Figure 3



TYPICAL STRESS-STRAIN CURVE FOR INDUSTRY-WIDE SELECTION OF GRADE 60 REINFORCING BARS

Figure 4

to 2 percent. Thus, there has not been an incentive to evaluate the use of welded wire fabric as the primary reinforcement of seismic rigid frames.

A recent series of bending tests by WJE for the Associated Reinforcing Bar Producers (ARBP)—Concrete Reinforcing Steel Institute (CRSI) on an industry-wide sampling of grade 60 reinforcing bars[9] indicated that currently produced grade 60 rebars have an excellent reserve ductility beyond the ductility exhausted in bends defined by minimum radii specified by ACI or ASTM. As expected, the strains exhibited in the bends considerably exceeded the strains exhibited in tension tests of the same bars.

Steel-industry research—In response to questions by code-writing bodies regarding the suitability of grade 60 steel in seismic structures, the steel industry has engaged in various specific activities. U. S. Steel developed a proprietary ductile weldable steel with a controlled composition, USS BEN-WELD, for use mainly in nuclear and seismic applications. This particular steel was eventually superseded by the new ASTM A706 specification[10] for rebars with a controlled composition, intended to be used where welding or bending, or both, are important.

Discussions within the technical subcommittee of the AISI Committee of Concrete Reinforcing Bar Producers led to the concept of specifying a lower limit on the as-produced ratio of rebar tensile strength to yield strength, rather than specifying a minimum tensile strength. This would ensure adequate strain-hardening capacity without letting the production be governed by the specification for the minimum tensile strength—a consideration that has frequently resulted in excessively high as-produced yield strengths. A tensile-yield ratio of only 1.25 was considered to be adequate. (It has been noted[11] that experimental research on A514 steel beams indicated that a tensile-yield ratio of only about 1.15 ensured enough strain-hardening capacity in steel wide-flange beams to permit the redistribution of moments required for plastic design.) As a result of these considerations, the AISI representative to the International Conference of Building Officials (ICBO) conservatively recommended* a specified minimum tensile-yield ratio of 1.33 for rebars. Although the 1.25 ratio was considered adequate, the 1.33 ratio was suggested because it was believed that currently produced grade 40 and grade 60 steels should consistently exhibit at least a 1.33 ratio.

In response to some comments that higher strength rebars were not as satisfactory as lower strength rebars that had a greater ductility, an ACI committee report[2] was prepared that compared grade 40, 60, and 75 steels. On the basis of the pertinent design limitations on steel percentages and on the ASTM requirements for ductility, the report demonstrated that the energy-absorption capacity of flexural members with the same static strength and geometry is about the same for all three grades of steel.

Related bond testing—There was concern that the bond characteristics of reinforcing bars could, under seismic loadings, result in slippage of the main longitudinal steel. Even if the steel itself had all the mechanical properties desired of a seismic rebar, the slippage could cause the flexural behavior of the reinforced concrete member to be as if the steel had a low modulus of elasticity and a "round-house" stress-strain curve. Therefore, various reversed-bending bond tests were conducted on specimens simulating beam-column joints in seismic frames. Published papers[12, 13] describing the tests conducted at the University of Texas did not identify any problem in seismic behavior that is related to the deformations on reinforcing bars. Consequently, there is currently no significant activity in the steel industry related to the deformations of deformed reinforcing bars. However, unpublished tests[14] conducted at the University of Washington have indicated some differences between the performance of reinforcing bars with different deformations in reversed-bending flexure tests of concrete beams where the reinforcing bar is stressed beyond yielding in both tension and compression.

Evaluation of prestressed concrete—There has been considerable discussion about the suitability of prestressed concrete members in seismic structures, but no extensive research has been aimed at evaluating the optimum characteristics of prestressing steel for that application. The 4 percent minimum elongation specified for prestressing wires[15] or bars,[16] and the

* At the April 29, 1971, meeting of ICBO at South Laguna, California.

3.5 percent minimum elongation specified by ASTM for seven-wire strand,[17] have been considered adequate by many experts.[18,19] It has been noted that the basic reason is that under seismic loadings the concrete will rotate extensively without cracking or crushing of the concrete and without a great increase in the steel strain,[20] yet with a significant increase in moment. This causes the portion of the flexural member that is subject to post-yield behavior to be generally greater for prestressed concrete than for nonprestressed concrete. The transformation from kinetic energy to potential energy thus involves significant storing of recoverable energy in the prestressed concrete, in contrast to significant absorption of nonrecoverable energy in the reinforcing steel of nonprestressed concrete.

Specifications for Seismic Reinforcing Steel

The past decade has witnessed some new specifications for the mechanical properties of reinforcing bars in seismic structures, but no specific seismic specifications for welded wire fabric or prestressing steel. ASTM A706 is a rebar specification[10] directed specifically toward the controls considered desirable for a nonprestressed seismic reinforcing steel—controlled composition to ensure strong ductile welds, both upper (78 ksi) and lower (60 ksi) limits on yield strength, and reduced (relative to A615[3]) diameters for bend tests. The minimum tensile strength is 80 ksi (552 MPa), instead of 90 ksi (620 MPa) for A615 Grade 60[3] steel, so that meeting the tensile-strength requirement does not result in a tendency to produce steel with a yield strength significantly above 60 ksi. For steel with this composition, the as-produced tensile-yield ratio can be expected to be satisfactory, in accordance with the previous discussion.

A706 steel is highly suitable for special structures where extreme safety precautions are demanded, such as nuclear containment structures. Therefore, there is currently (March 1977) an ASME code case (No. 1784) in which the reply to the official inquiry is that it is the opinion of the ASME Boiler and Pressure Vessel Committee that reinforcing material conforming to ASTM A706-75[10] may be used in the construction of concrete reactor vessels and containments. However, A706 steel is currently not a significant item in general construction because of its limited availability. Usually, it is produced only to special order, and must be purchased in heat quantities. The premium (typically at least $50 to $75 per ton or more over the cost of A615 Grade 60 steel) that must be paid for A706 steel discourages the specification of the steel by architects and engineers and provides no incentive for stocking the steel. Consequently, the steel industry has emphasized that currently produced A615 grade 60 steel[3] generally has all the qualities desired for seismic bars, except that welding may not be practical for the end connection of large bars. That problem has been solved by the development of reliable mechanical splices. The steel industry points out that the two series of tests by WJE on industry-wide samplings of A615 grade 60 reinforcing bars[7,9] have shown these rebars to consistently exhibit a well-defined yield point, a yield point generally below 78 ksi, and excellent ductility.

The primary organization for originating seismic specifications has been the Structural Engineers Association of California (SEAOC). As a result of the steel-industry activity, the SEAOC Recommended Lateral Force Requirements and Commentary[21] specifies that the as-furnished yield strength of grade 60 rebar may not exceed 78 ksi, and that the tensile-yield ratio may not be less than 1.33. The SEAOC requirements for seismic rebars are generally adopted by ICBO for the Uniform Building Code.[22]

Practical Production Limitations

Questions are frequently asked regarding the reluctance of the steel industry to readily agree with the restrictive mechanical-property limitations that are considered desirable for seismic reinforcing steels. The problem has been that the engineers who have been formulating codes have not been fully aware of the problems of producing steels to narrow specification limits. One example was mentioned above; meeting the tensile-strength requirement has had a tendency to raise the yield strength significantly above the specified minimum value. It would thus be logical to relax the tensile-strength requirement somewhat to obtain a more consistent yield strength. However, there still is the problem of producing different size bars from the same heat of steel. Larger diameter bars cool slower and therefore have lower strengths when produced from steel with the same composition. Also, there is invariably some difference in composition throughout the heat, that is, a difference between the top-cast, middle, and bottom-cast portions.

Consequently, in arriving at seismic specifications and special requirements for seismic structures, both the code writers and the steel industry have compromised. Seismic codes permit as much as 18-ksi (124 MPa) variation in yield strength.[21,22] However, currently produced reinforcing bars have been improved to the extent that as mentioned above, a well-defined yielding is generally obtained with grade 60 reinforcing bars, and the bendability of these steels is superior to the ASTM and ACI bend-test requirements. The individual steel companies have expended large sums of money on research and production control to achieve these goals, which should help to provide an adequate safety margin for structures reinforced with currently available steels. Consequently, the steel industry has become disillusioned with comments from members of code-writing bodies which indicate that specifications should be made more restrictive as a result of these improvements in the mechanical properties of the reinforcing steels. This applies to the attempt by members of code-writing bodies to promote a grade 80 (80-ksi or 552 MPa specified minimum yield strength) reinforcing bar, which would be expected to have a well defined yield point, even though that is practically impossible with the alloy grade of steel that would be necessary to consistently meet an 80-ksi yield strength. It also applies to recent suggestions that because of the favorable WJE tests,[9] the minimum permissible bend radii be decreased to facilitate detailing of complex structures. It thus appears that code writers want increased ductility along with increased strength. They appear to forget that increased strength is generally obtained at the expense of decreased ductility.

PROPOSITIONS RESULTING FROM A REEVALUATION OF THE REQUIRED SEISMIC PROPERTIES OF REINFORCING STEELS

What direction should future activities take relative to specifying or obtaining reinforcing bars with specific mechanical characteristics aimed at optimizing the structural response to an earthquake? Before answering that question, it is necessary to reevaluate what is really desirable for the shape of the tensile engineering stress-strain curve of the steels. Most critical is the main longitudinal steel of flexural members, because that is the steel which must absorb energy inelastically in seismic design or, in the case of prestressing steel, must maintain compression in the concrete as it rotates extensively and stores energy. Therefore, the following remarks apply to the regions of plastic hinging in flexural members. A series of propositions is derived, which are subject to the critical scrutiny of the Workshop participants. Reinforcing bars and prestressing steel are treated separately because their function is generally different during the plastic hinging.

Reinforcing Bars

Required ductility and definition of yielding—In nonprestressed concrete, the required ductility is that which is necessary and sufficient to ensure ductile failure of the structure; that is, a structure in which at least the main longitudinal tensile steel extends inelastically a significant amount, without rupturing, before the concrete crushes. This leads to

> Proposition No. 1 - the useful ductility of the main longitudinal reinforcing bars in a flexural member need not exceed the tensile strain at which the concrete on the other side of the member begins to crush.

By using the principles of statics and strain compatibility, Equation C-4 is developed in Appendix C for relating the condition of concrete crushing to corresponding tensile-steel stresses and strains. The derivation does not require any assumption for the shape of the stress-strain curve. Therefore, tensile-steel stress-strain sets apply equally well to stress-strain points on a yield plateau, in the strain-hardening range, or on a roundhouse stress-strain curve.

It is instructive to determine what the general magnitude of steel strains (that is, useful ductility) should be at different steel stresses if there is not to be premature concrete crushing. For the numerical evaluations, consider (1) concrete with $f_c' = 4000$ psi (276 MPa) and the usually assumed crushing strain of 0.3 percent, (2) designs having different ratios of $(\rho - \rho')$ to ρ_b where the balanced-design steel percentage, ρ_b is computed[20] for a specified yield strength of 60 ksi, and (3) steels meeting this specified yield strength but having different stress-strain curves and thus different stresses corresponding to a given post-yield strain. The result of using Equation C-4 is shown in Table I, which gives, for various values of the parameters, the theoretical steel tensile strain at which the concrete on the other side of the member begins to crush.

Table I demonstrates the well-known fact that for a given percentage of reinforcing steel, greater steel strains are obtainable without concrete crushing when the tensile stress is less. With relation to Table I, the plots in the Appendix B figures, which typically exhibit stresses between about 70 ksi (483 MPa) and 100 ksi (689 MPa) at a 2.5 percent strain, indicate that generally less than 4 percent tensile-steel strain can be experienced before concrete crushing when A615 grade 60 steel[3] is used. This

Table I

Tensile-Steel Stress-Strain Condition Corresponding to Concrete Crushing on Other Side of Flexural Member

$f_c' = 4000$ psi   Specified $F_y = 60$ ksi

Balanced-design percentage, ρ_b, computed for $F_y = 60$ ksi; ρ and ρ' are the tension and compression steel percentages, respectively.

$\frac{\rho - \rho'}{\rho_b}$	Value of ε, in./in.		
	$f_s = 60$ ksi	$f_s = 78$ ksi	$f_s = 90$ ksi
0.75	0.00376	0.00220	0.00151
0.50	0.00714	0.00480	0.00376
0.25	0.01728	0.01260	0.01052
0.117*	0.04032	0.03033	0.02588

* Based on $\rho_b = 0.0285$, $\rho = 2\rho'$ to reflect the requirement that the positive moment capacity of flexural members at column connections shall not be less than 50 percent of the negative capacity, and $\rho' = 200/F_y$, which is the lower limit, ACI 318-71.

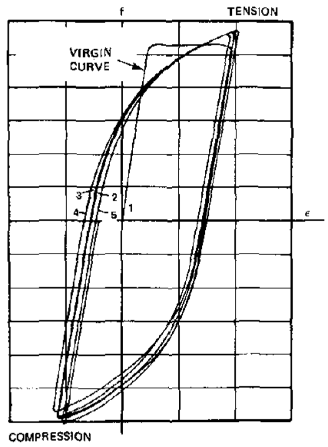
Conversion Factors

1 psi = 6.895 kPa
1 ksi = 6.894 MPa

demonstrates the reason for imposing an upper limit on the yield strength of A706 steel and the reason that it is desirable to have some significant yielding before appreciable strain hardening. However, these calculations reveal no reason for requiring a well-defined yield point other than to control the ascent of the stress-strain curve beyond yielding. This leads to

> Proposition No. 2 - for the tensile reinforcing steel to provide an optimum seismic response, the yield strength need not be well defined at or before any specific strain, but a round-house stress-strain curve is not preferred, and there should be some significant inelastic straining before the onset of appreciable strain hardening.

Bauschinger effect—An earthquake involves repeated reversed movements of the structure. During the cycles of inelastic straining following initial inelastic straining, the main longitudinal tensile steel will have a round-house stress-strain curve generally below the virgin curve in the early portion of the yield region but slightly above the virgin curve in the later portion of the yield region. This is the Bauschinger effect, Figure 5.[24] Because the Bauschinger effect results in some increase in stress (over that in the virgin material) for larger strains, proposition No. 2 leads to



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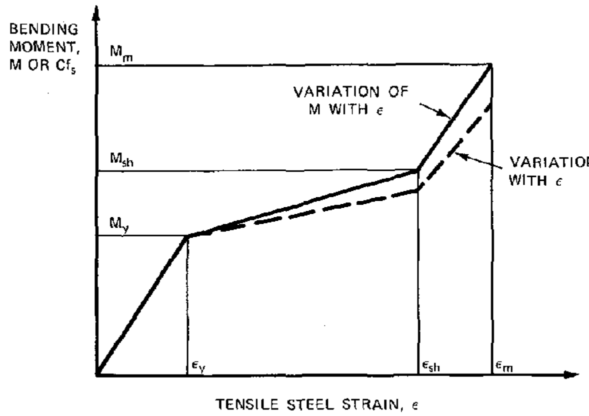
BANSCHINGER EFFECT EVIDENCED IN REPEATED REVERSED BENDING OF A REINFORCING STEEL BAR

Source: A. Singh, K. H. Gerstle, and L. G. Tulin, "The Behavior of Reinforcing Steel Under Reversed Loading," Materials Research and Standards, January 1965.

Figure 5

Proposition No. 3 - the Bauschinger effect should be considered in determining the conditions required to ensure steel yielding without concrete crushing in seismic structures.

Strain-hardening requirements—To evaluate the strain-hardening requirements of seismic reinforcing bars, it is necessary to consider the changing moment gradient within the portion of the flexural member where the tensile steel extends inelastically, Figures 2 and 6. The steel yielding starts at one spot and then extends over a finite length as the moment gradient increases. The bending moment will vary from the moment at first yielding of the tensile steel, $M_y = Cf_y$, to M_m, which is the maximum moment considered in the present discussion. (C is a constant and f_y is the steel yield strength.) The corresponding steel strain, at any distance along the member, can be obtained if the plot of resisting moment vs steel strain is known, Figure 6. Beyond yielding of the tensile steel, the increase of the moment above M_y results from two effects: (1) strain hardening, which is the increase of steel stress, f_s, with increase of strain, reflected in the slope of the dashed curve in Figure 6, and (2) the shape factor, S, which is reflected by the



M_y = MOMENT AT FIRST YIELDING OF TENSILE STEEL
ϵ_y = STRAIN AT FIRST YIELDING OF TENSILE STEEL
M_{sh} = MOMENT AT ONSET OF STRAIN HARDENING OF TENSILE STEEL
ϵ_{sh} = STRAIN AT ONSET OF STRAIN HARDENING OF TENSILE STEEL
C = M_y/f_y
f_y = STEEL YIELD STRENGTH
f_s = STEEL STRESS

VARIATION OF MOMENT WITH STRAIN

Figure 6

vertical distance between the dashed and solid curves. The shape factor is the ratio of the moment arm of the resisting moment divided by the moment arm of the moment at first yielding, and reflects the changing shape of the concrete compression stress block and the changing stress in the compression steel. The shape factor can be determined approximately as the ratio of the ultimate-stress-design (USD) moment[23] (which in accordance with the ACI Building Code is based on the steel yield strength) to the moment at first yielding.

Table II

Shape Factors Computed as USD
Moment Divided by Moment at First Yielding

f'_c = 4000 psi   Specified F_y = 60 ksi $d'/d = 0.1$

Balanced-design percentage, ρ_b, computed for F_y = 60 ksi; ρ and ρ' are the tension and compression steel percentages, respectively.

		Shape Factor	
ρ	ρ'	Actual f_y = 60 ksi	Actual f_y = 78 ksi
$0.75\ \rho_b = 0.0214$	0.5ρ	1.046	1.049
$0.50\ \rho_b = 0.0143$	0.5ρ	1.038	1.044
0.0214	zero	1.007	1.003
0.0143	zero	1.015	1.006

Conversion Factors
1 psi = 6.894 kPa
1 ksi = 6.894 MPa

To allow the desirable extension of the region of plastic hinging (the distance between X_1 and X_3 in Figure 2) the moment gradient must be accommodated within that region. This can be accomplished by having either adequate strain hardening, an adequate shape factor, or a combination of both. Appendix D presents the procedure for determining the shape factor as the ratio of the USD moment to M_y. This was applied to evaluations with f_c' = 4000 psi and with either no compression reinforcement or with reinforcement near the compression surface having half of the tensile-steel area. The results, Table II, indicate that the shape factor can be as high as about 1.05 if longitudinal compression steel is present and highly stressed, or only about 1.0 if only the tensile steel is present or if the compression steel is not highly stressed. The compression steel will be more highly stressed for smaller ratios of d'/d, where d' and d are the distance from the compression face of the concrete to the compression and tension steels, respectively. Because it is not practical to set limits on d'/d, it must be assumed that in some designs the shape factor may be only about 1.0. Therefore, Proposition No. 4 follows:

Proposition No. 4 - to ensure accommodation of the moment gradient within the plastic hinge, specifications for 60-ksi-yield reinforcing bars should require the minimum tensile strength to be at least 80 ksi or the tensile-yield ratio to be at least 1.25.

Ramp vs yield plateau—To provide maximum inelastic straining of the tensile reinforcing steel within the region of plastic hinging, there will be some optimum shape of the stress-strain curve. Referring to Figure 6, the total elongation of the tensile steel within the region between X_1 and X_3 is

$$e = \left(\frac{\varepsilon_y + \varepsilon_{sh}}{2}\right)\left(X_3 - X_2\right) + \left(\frac{\varepsilon_{sh} + \varepsilon_m}{2}\right)\left(X_2 - X_1\right) \qquad (1)$$

The straining is maximized by maximizing ε_{sh}, ε_m, and $(X_2 - X_1)$, because the average strain over $(X_2 - X_1)$ is greater than the average strain over $(X_3 - X_2)$. For a given range of $(M_m - M_{sh})$ and hence of $(X_3 - X_1)$, it is seen in Figure 2 that maximizing $(X_2 - X_1)$ implies minimizing M_{sh}. This implies that between yielding and strain hardening a plateau or ramp-shape stress-strain curve, Figure 1 or 4, would be preferable to a round-house stress-strain curve for maximizing the plastic-hinge capacity. Thus, there follows

Proposition No. 5 - after yielding, inelastic straining of the tensile steel will be maximized by a plateau or ramp-shape stress-strain curve rather than by a round-house stress-strain curve.

Prestressing Steel

The ACI code provisions for prestressed concrete ensure that the prestressing steel will undergo some inelastic action before the concrete crushes. This is accomplished by specifying that the appropriate sum of the reinforcing indices (that is, the prestress-steel reinforcing index plus the tension, nonprestress steel index minus the compression nonprestress steel index) does not exceed 0.3.[23] Each reinforcing index is the steel percentage times a steel stress divided by f_c'. The steel stress is f_y for the nonprestress steel or f_{ps} for prestressing steel; f_{ps} is the calculated stress at the design load, and is permitted to be as great as 0.7 f_{pu}, where f_{pu} is the ultimate strength of the prestressing steel.

The effectiveness of this restriction in ensuring some significant inelastic action can be gained from a consideration of the usual equation for balanced design[23] (simultaneous steel yielding and concrete failure). With the equation generalized to apply to any set of values of steel percentage, ρ, and concrete stress, f_s, concomitant with concrete crushing, it may be written as

$$\frac{\rho f_s}{f_c'} = 0.85\ \beta_1 \left(\frac{0.003}{0.003 + \varepsilon_s}\right) \qquad (2)$$

where $\beta_1 = 0.85$ for $f_c' \leq 4000$ psi and ε_s is the steel strain corresponding to f_s. Assume, as a critical condition for the prestress steel, that $f_s = f_{p\mu} = f_{ps}/0.7$ and $\varepsilon_s = 0.002$. Equation 2 then becomes

$$\frac{\rho f_{ps}}{f_c'} = 0.3 \tag{3}$$

which corresponds to the ACI restriction. Thus, with designs limited by this restriction, the prestressing steel should strain at least 2 percent before the concrete begins to crush.

For earthquake-resistant structures, it could also be desirable to specify a lower limit on the prestress reinforcing index so that the steel would not rupture before the concrete crushes. Prestressing steel generally exhibits a ductility of at least 4 percent elongation[18], which is required by ASTM for wires[15] and bars[16] and which is generally met by the seven-wire strand, even though ASTM requires[17] only 3.5 percent for the strand. Substituting 4 percent for ε_s in Equation 2 and using $f_s = f_{p\mu} = f_{ps}/0.7$ would then give

$$\frac{\rho f_{ps}}{f_c'} = 0.22 \tag{4}$$

It is obviously not practical to restrict designs so that

$$0.22 \leq \frac{\rho f_{ps}}{f_c'} \leq 0.30$$

Such a lower limit is not given in the ACI Code. Instead, the Code gives a lower limit based on crack control[25]. Nevertheless, there follows

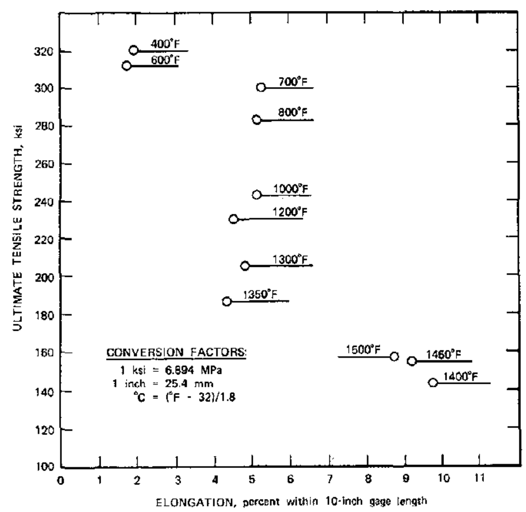
> Proposition No. 6 - in the seismic response of a prestressed flexural member, a 4 percent ductility is adquate for maximum utilization of the prestressing steel where it provides a reinforcing index between about 75 percent and 100 percent of the maximum index permitted by the ACI Code; for a lesser index, a greater steel ductility could be utilized.

FUTURE WORK

The six propositions stated above are subject to critical examination. Suggestions for future research should be made where disagreements regarding the propositions cannot be resolved or where future questions arise from discussions of the propositions.

Ductility is considered to be the most important attribute for seismic reinforcing steel, and strain-hardening capacity is considered to be important for reinforcing bars. However, the needs to develop steels with greater ductility and/or strain-hardening capacity than those currently available, or the needs to make comparative tests of concrete members reinforced with steels having different ductilities and/or strain-hardening capacities are doubtful. Currently available A615 or A706 reinforcing bars appear to have sufficient ductility for flexural members, that is, over 4 percent measured at the high point on the engineering stress-strain curve. For a greater reinforcing steel ductility to be utilized in nonprestressed flexural members, the concrete crushing strain must be increased considerably above 0.3 percent. Research aimed toward the goal would involve determining practical levels of concrete confinement.

There are some structures in which improved reinforcing-steel ductility would be beneficial. A seventh proposition could state the obvious fact that the response of a structure stressed entirely in tension would be optimized by any increase in the steel ductility. However, such a structure would be unusual, and the development of special steels for membrane-tension or axial-tension structures would not appear warranted. Prestressed concrete structures could possibly be optimized to a somewhat greater extent if prestressing strand with a greater ductility could be obtained. However, research has indicated that increased ductility of prestressing wire or strand can only be obtained by stress relieving at a higher temperature, which lowers the steel tensile strength, as indicated in Figure 7.



COURTESY OF U. S. STEEL CORPORATION

ULTIMATE TENSILE STRENGTH AND DUCTILITY OF 0.096-INCH-DIAMETER WIRE (0.80%C, 0.70%Mn) STRESS-RELIEVED AT VARIOUS TEMPERATURES

Figure 7

Instead of developing and then testing some new steel with greater ductility or greater strain-hardening capacity, it would be more appropriate to make more refined experimental evaluations of what actually occurs within the plastic-hinging regions of both nonprestressed and prestressed flexural members subjected to the distortions anticipated during an earthquake. Specifically, strain-gage measurements could determine the strains that actually occur in the steel, thus leading to an evaluation of the extent to which ductility of the steel is really utilized. Also, strain-gage and/or extensometer measurements could provide information for evaluating the length of the plastic hinging. With that information and a knowledge of the loading and hence the moment gradients, the increase in moment occurring over the plastic hinge can be determined. From this increase, the requirements for strain-hardening capacity of the steel can be evaluated.

CONCLUSIONS

As a result of previous work and the analyses in the present paper, six propositions have been stated relative to defining specific mechanical properties of reinforcing steels so that, with reinforcing bars and prestressing steel having such properties, the response of concrete flexural members to an earthquake will be optimized. Generally, the ductility and strain-hardening capacity of currently available reinforcing bars and prestressing steels appear sufficient for most applications. However, this is subject to question for specific applications as are all of the six listed propositions. A greater steel ductility could be justified only if improved concrete confinement could result in significantly greater concrete crushing strain. Fundamental unanswered questions include quantitative determinations of what inelastic steel strains and moment variations can be expected to occur within the plastic-hinge regions of rigid-frame girders and shear walls that are subjected to severe earthquakes.

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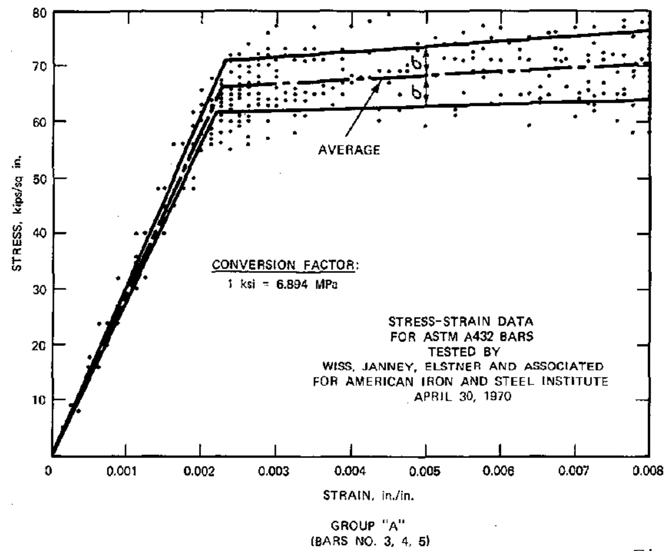


Figure A-1

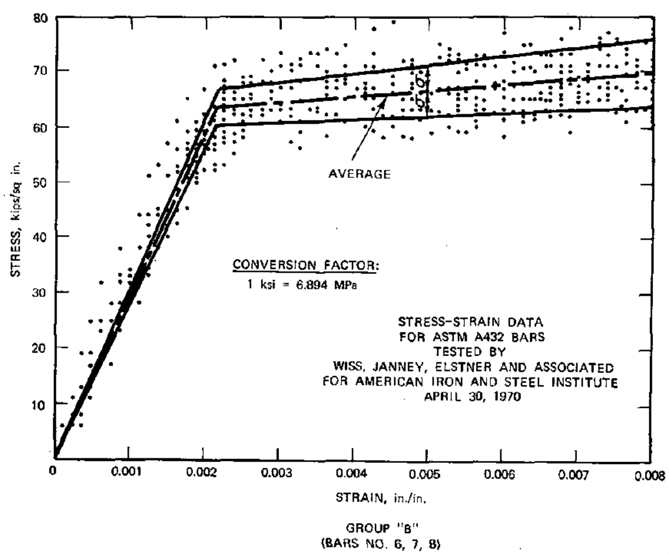


Figure A-2

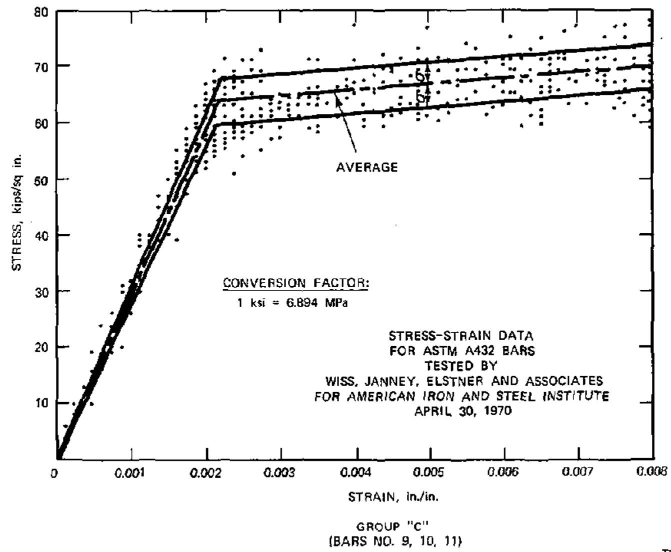


Figure A-3

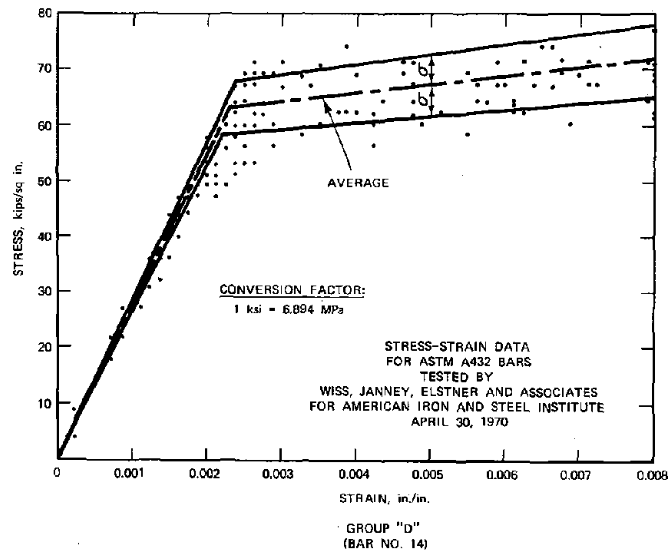
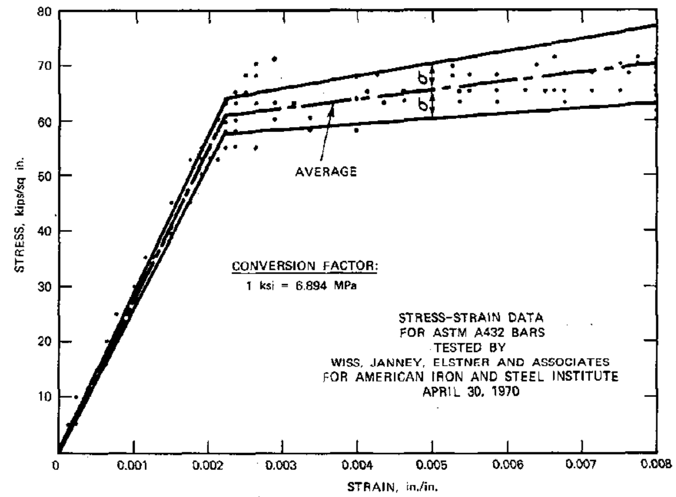


Figure A-4



GROUP "E"
(BARS NO. 18)

Figure A-5

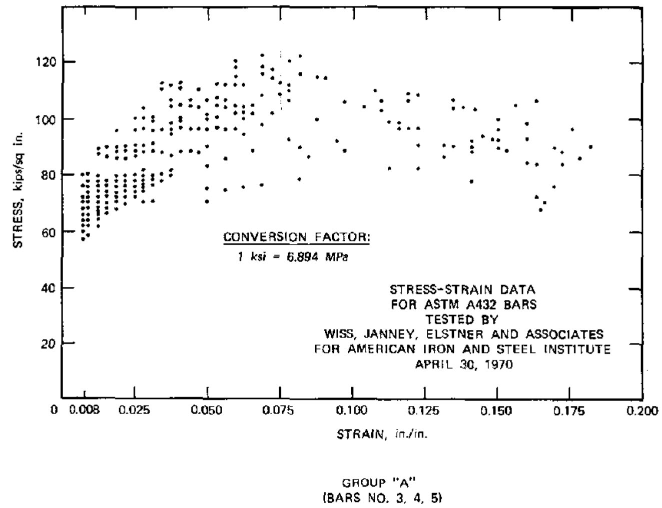


Figure B-1

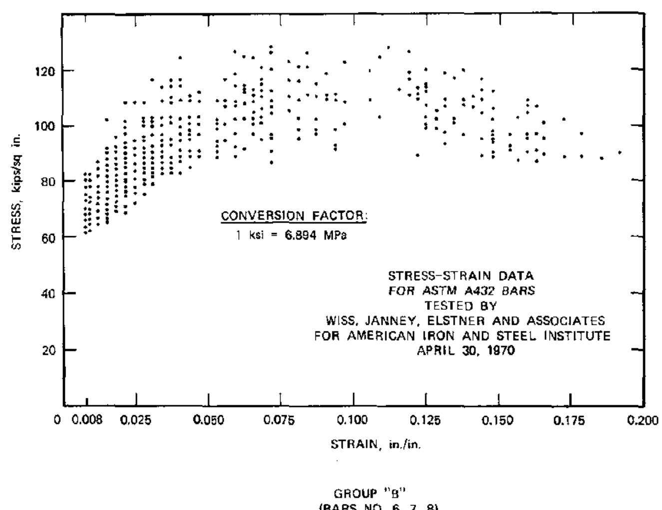


Figure B-2

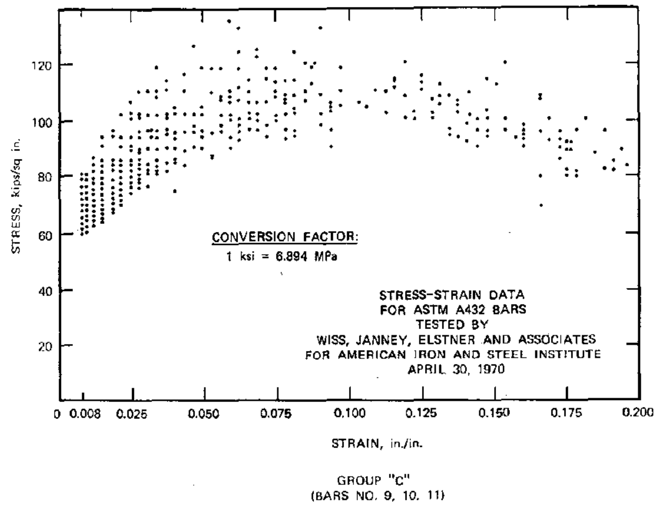


Figure B-3

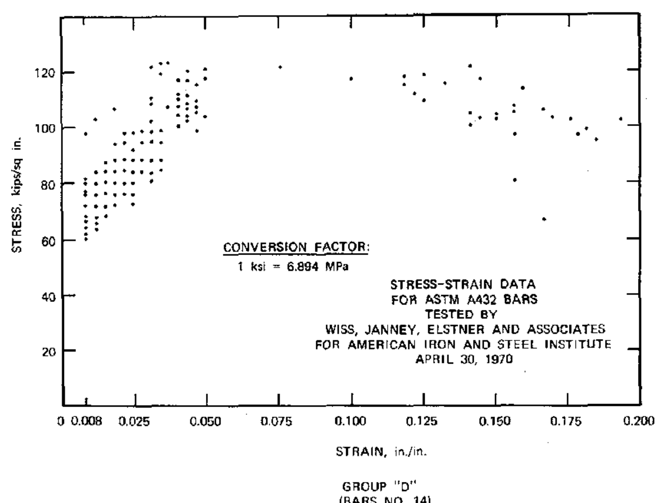
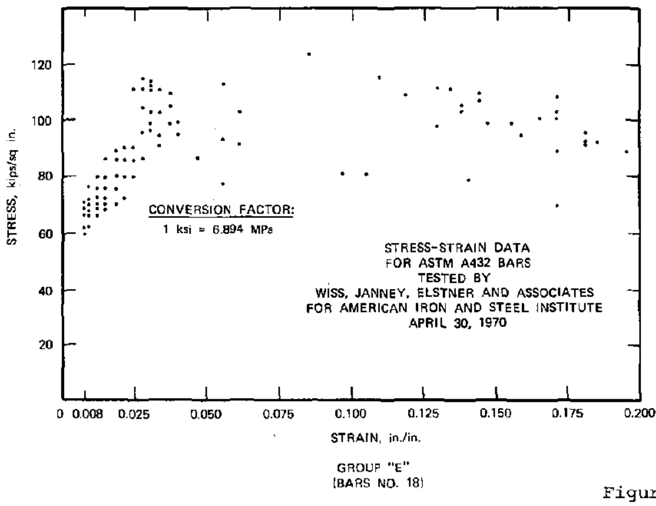


Figure B-4

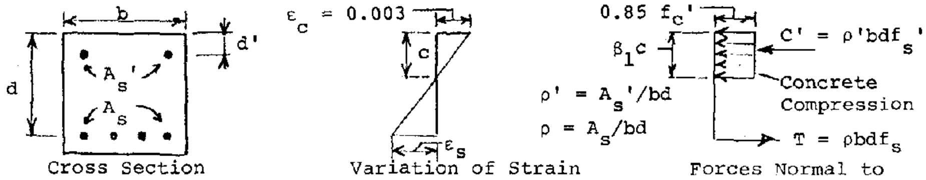


120
100
80
60
40
20
STRESS, kips/sq in.
0 0.008 0.025 0.050 0.075 0.100 0.125 0.150 0.175 0.200
STRAIN, in./in.
CONVERSION FACTOR:
1 ksi = 6.894 MPa
STRESS-STRAIN DATA
FOR ASTM A432 BARS
TESTED BY
WISS, JANNEY, ELSTNER AND ASSOCIATES
FOR AMERICAN IRON AND STEEL INSTITUTE
APRIL 30, 1970
GROUP "E"
(BARS NO. 18)

Figure B-5

Appendix C

Calculation of Tensile-Steel Stress-Strain Condition Corresponding to Concrete Crushing on Other Side of the Flexural Member



Cross Section of Member — Variation of Strain on Cross Section — Forces Normal to Cross Section

f_s and f_s' are the tensile and compression steel stresses; f_c' is the concrete compression strength. Equilibrium of the forces on the cross section gives

$$\rho bdf_s = b\left[\beta_1 d\left(\frac{0.003}{0.003 + \varepsilon_s}\right)\right](0.85\ f_c') + \rho' bdf_s' \qquad \text{(C-1)}$$

Assume $f_s \leq f_y$ and d'/d is small so that it can be considered that $f_s' = f_s$. Then, Equation C-1 becomes,

$$\varepsilon_s = 0.003\left[\frac{0.85\ \beta_1 f_c'}{(\rho - \rho')f_s} - 1\right] \qquad \text{(C-2)}$$

Let $K_1 = (\rho - \rho')/\rho_b$ and $K_2 = f_s/F_y$ where the balanced design reinforcing percentage is

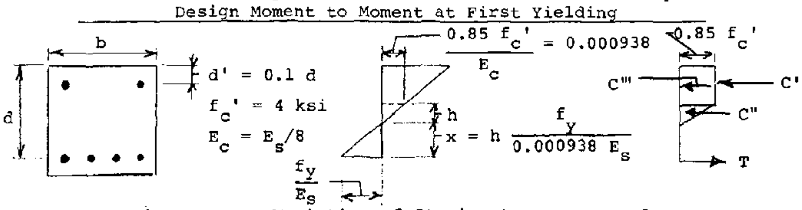
$$\rho_b = \frac{0.85\ \beta_1 f_c'}{F_y}\left(\frac{0.003}{0.003 + F_y/E_s}\right) \qquad \text{(C-3)}$$

$$= 0.0285 \text{ for } f_c' = 4 \text{ ksi and } F_y = 60 \text{ ksi.}$$

F_y is the specified yield strength and E_s is the modulus of elasticity, 29,000 ksi. Then,

$$\varepsilon_s = \frac{1}{K_1 K_2}\left(0.003 + \frac{F_y}{E_s}\right) - 0.003 \qquad \text{(C-4)}$$

Calculation of Shape Factor as Ratio of Ultimate-Strength-Design Moment to Moment at First Yielding



Cross Section of Member — Variation of Strain at First Yielding — Normal Forces at First Yielding

At first yielding,

$$M_y = (C')(0.9\ d) + (C")(x + \frac{2h}{3}) + C'''\ (\frac{d + h + x}{2}) \tag{D-1}$$

where

$$C' = (\rho' bd)\left[(0.000938\ E_s)(\frac{0.9d - x}{h})\right] = 27.2\ \rho' bd\ (\frac{0.9d - x}{h}) \tag{D-2}$$

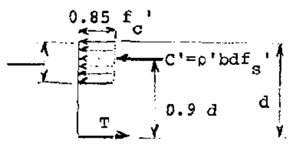
$$C" = (0.5\ bh)(0.85\ f_c') = 1.7\ bh \tag{D-3}$$

$$C''' = (d - h - x)b\ (0.85\ f_c') = 3.4\ b\ (d - h - x) \tag{D-4}$$

$$C' + C" + C''' = T = \rho bdf_y \tag{D-5}$$

With x defined in terms of h and ρ', ρ, and f_y determined, substituting Equations D-2, D-3, and D-4 into Equation D-5 results in the solution of h/d and hence x/d, which leads to evaluating Equation 1 as M_y = a constant times bd^2.

$$a = \frac{(\rho f_y - \rho' f_s')d}{0.85\ f_c'}$$



Ultimate-Strength-Design Normal Forces

With f_s' assumed to be f_y, the ultimate-strength-design moment is

$$M_u = (ab)(0.85f_c')(d - \frac{a}{2}) + 0.9\ \rho' bd^2 f_y \tag{D-6}$$

$$S = \text{shape factor} = M_u/M_y \tag{D-7}$$

MECHANICAL CHARACTERISTICS AND BOND OF REINFORCING STEEL UNDER SEISMIC CONDITIONS

by

Egor P. Popov
Professor of Civil Engineering
University of California, Berkeley

INTRODUCTION

For decades the design of reinforced concrete structures has been based on the assumption of linearly elastic behavior, and this approach at the frame analysis level continues until today. Very gradually, the era for determining the deimensions of cross-sections and their reinforcement on the basis of ultimate strength design has arrived [1]. The latter approach clearly recognizes the ductile properties of steel beyond yield and correctly exploits the useful inelastic range of concrete behavior. This procedure assumes monotonic or unidirectional application of loads which causes progressive yielding of steel with the associated formation of cracks in the tension zones. As the intensity of load increases, the steel continues to yield and the number and width of cracks increases until the concrete which was in compression from the outset finally crushes.

Critical reinforced concrete members employed in resisting seismic loading behave entirely differently. The essential difference is due to the fact that intense loadings reverse in direction and usually are cyclic in their character. This dramatically changes the behavior of a reinforced concrete member. For severe loadings, such as occur in a major shake, cracks first form on one side of a member and then on the other, i.e., the order of tension and compression zones changes. Moreover, if the intensity of load application is sufficiently great, the strained steel prevents the cracks from ever closing. In this manner, cracks penetrate through the whole depth of a member and forces are carried entirely by steel. This being the case, it is very important to have good knowledge of the behavior of steel alone under repeated and reversed loading conditions. Since in cases where the cracks do close the concrete picks up the compressive forces, in the study of cyclic behavior of steel a rather limited range of strain into the compressive zone need be explored. This differs from the problem encountered in structural steel frames. Further, although at an open crack itself, it is clear that reinforcing steel carries the force, beyond the crack this force must be transferred to concrete. This gives rise to the bond problem which becomes particularly complex under cyclic loading. Deterioration in bond due to cyclic loading requires especial attention.

This paper addresses itself to two of the above problems. First, the available mathematical models for inelastic cyclic behavior of reinforcing steel are discussed. This includes some comments on the behavior of prestressing steel in aseismic design. Second, the bond behavior of reinforcing steel under load reversals is considered. The substantial gap in knowledge in this area is indicated, and a need for bond-slippage law is emphasized. This kind of information on steel reinforcement together with mathematical formulation of the constitutive relation for concrete under multiaxial state of stress is essential for further developments in rational analysis of reinforced concrete members and structures.

A sharp distinction in the needs for mathematically describing the above phenomenae for research and design must be always kept in mind. In research, detailed comparisons of experimental results with predictions of the behavior of test specimens for which the properties of materials are accurately known must be made for the purposes of clearly understanding the physical mechanisms involved. In design, with considerable variations likely in the actual strengths of both concrete and steel, a solution must provide only basic insight into the overall behavior of a structure. Moreover, only direct short-cut methods are practical.

CONSTITUTIVE RELATIONS FOR STEEL

Constitutive relations for metals and for steel in particular have received a good deal of attention in recent years. In many current engineering applications such relations are needed for multi-axial state of stress and a substantial amount of literature on the subject sprung up [2,3,4,5 and 6]. In reinforced and prestressed concrete interest is confined to an uniaxial condition. Nevertheless, since the approaches developed for the more general cases of stress readily reduce to that of the uniaxial case, such broader formulations must be examined for possible use for reinforcing steel.

The relatively simple monotonic uniaxial stress-strain behavior of steel is well understood. It is recognized that 40 grade steel has a long plastic plateau; whereas, the plateau for 60 grade steel is very short, and for prestressing steel, it is non-existent. Any number of schemes are available to represent the continuous curves and the strain-hardening range for ordinary steel (inclined straight lines, parabolas, Hermitian functions, etc.). For ultimate strength design the writers of the ACI Code [1] chose the simplest alternative of an elastic-perfectly plastic relationship, which ignores the strength increase due to strain-hardening. For monotonic loading this is a good conservative choice. In seismic design, which deals with a highly nondeterministic problem and repeated and reversed loading conditions of great severity, so simple an approach cannot be adopted. In order to achieve a balanced design, the ultimate strengths of members and joints must be determined more accurately. A weak link in the structural chain may have dire consequences. Hence, a simple bi-linear stress-strain relation for repeated and reversed loadings will not do, and more accurate formulations must be used.

The available formulations for stress-strain relations for cyclic behavior can be conveniently classified into two groups. One of these groups is based on generalizations of the Ramberg-Osgood equations. The other approaches display considerable differences in their formulation, but all consider history dependence in the material behavior. These will be referred to as history dependent formulations. Details of these approaches follow.

Generalized Ramberg-Osgood Formulations

Among researchers in reinforced concrete, by far the most widely used procedure of formulating the random cyclic behavior of steel is based on

Ramberg-Osgood (R-O) equations. No applications to cyclic behavior was envisoned in the original paper. What actually happened was that the R-O equation was used together with the Masing hypothesis [7] published earlier. The original Masing hypothesis asserts that an initial monotonic curve, such as given by the R-O equation, when magnified by a factor of two, will define the hysteretic loop shape of any branch when the origin of this new curve is placed at the point of stress reversal. This approach was successfully applied by a number of investigators in studying high-cycle behavior of metals [8]. However, for low cycle representations with very large inelastic strain reversals a number of modifications are necessary, and it is this kind of formulation that is essential in seismic analyses.

First of all, a curve for the monotonic loading the material well into the strain-hardening range must be available. Then the reversals are made using the R-O equation. Among the papers using this approach, the following may be mentioned: Aktan, Karlsson, and Sozen [9], Kent and Park [10], and Ma, Bertero, and Popov [11 and 12]. All of these investigators show excellent agreement of the predicted hysteretic loops with the experimental ones. An example of the type of agreement found by Aktan, et. al., is shown in Fig. 1; by Kent and Park in Fig. 2. In this paper, the procedure developed by Ma et. al

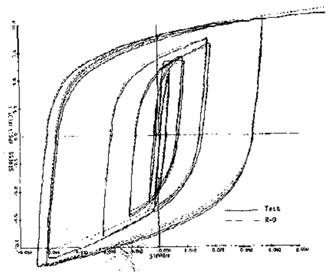


Fig. 1. Comparison of R-O Model with Experimental Results [9)

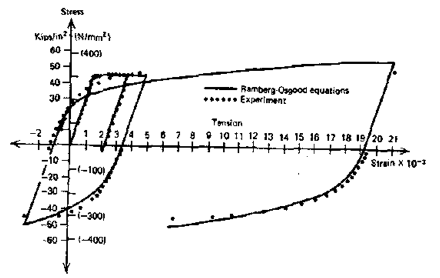


Fig. 2. Stress-Strain Curve for Steel with Cyclic Loading [10]

will be discussed in detail. This procedure described in References 11 and 12 employs the Ramberg-Osgood function to describe the Bauschinger effect of strain-softening under reversed loadings; a separate set of rules is used to define the cyclic strain-hardening behavior. In this procedure an ordinary σ-ε curve under monotonic loading is sufficient to define cyclic material properties. The seven basic points required to define such a curve are shown in Fig. 3(a).

Two possible cases of first stress reversals are shown in Fig. 3(b); one of them occurs in the plastic plateau Range (Point A), the other in the strain-hardening range (Point A'). For the latter case, upon unloading, the stress is first reduced elastically from A' to A". The σ-ε relationship between A and B or A" and B' is given by a Ramberg-Osgood equation:

$$\bar{\varepsilon}_S = \beta \frac{\bar{\sigma}_S}{|\bar{\sigma}_S|} \left(|\bar{\sigma}_S| + \alpha\, |\bar{\sigma}_S|^n \right) \qquad (1)$$

where

$$\bar{\varepsilon}_S = (\varepsilon_S - \varepsilon_{SA})/2\varepsilon_y$$

$$\bar{\sigma}_S = (\sigma_S - \sigma_{SA})/2\sigma_y$$

in which ε_S and σ_S define a point on AB (or A"B'), and ε_{SA} and σ_{SA} define point A (or A").

For an accurate representation of the reversal curves, the parameters α, β, and n must be varied depending on the magnitude of the residual plastic strain, ε_{Pmax}, which would develop upon release of the previous loading. In Fig. 3(b) this strain corresponds to the distance SO for Point A, and SO' for A'. For the materials used in this investigation, α and β can be determined empirically by

$$\alpha = 2.3\ \varepsilon_{Pmax}/\varepsilon_{sh} < 2.3$$

$$\beta = [1 + 0.7\ \varepsilon_p - 0.3\ \bar{\varepsilon}^{7/3}] < 1.4$$

where

$$\bar{\varepsilon}_p = \varepsilon_{Pmax}\ /(35 \times 10^{-3})$$

For the two possible cases described above, n is 6 and 7, respectively.

Beyond points such as B or B', the σ-ε relationship is assumed to be given by the rotated and translated monotonic strain-hardening curve, such as CY in Fig. 3(a).

If loading reverses a second time, similar to that shown at Point E in Fig. 3 (c), the σ-ε relationship of the ascending curve EF is determined from the shape of the previous descending curve AE. For example, the curve EF is established by rotating the curve AE through 180° and translating it so that Point A coincides with E. This procedure is applicable provided ε_{Pmax} for Point F is equal to or less than the corresponding quantity for Point A. For loading beyond Point F, the σ-ε relationship is assumed to follow the original monotonic σ-ε curve. The reversal at G is based on the parameter ε_{Pmax} corresponding to this point.

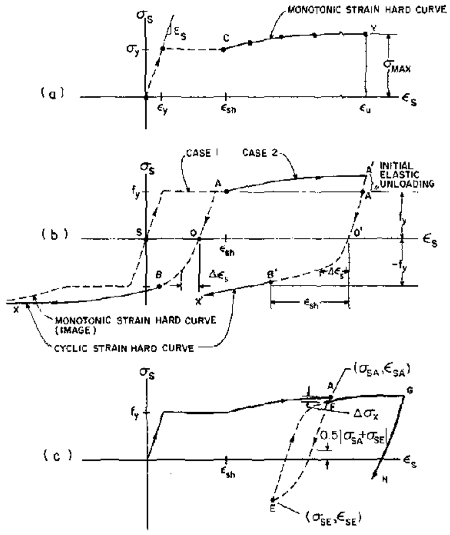


Fig. 3. Construction of Analytical Hysteretic σ-ε Relationship of Reinforcing Steel [11].

For an accurate modeling of the initiation of cyclic strain-hardening in the plastic plateau range, two cases must be differentiated. The quantity 0.5 $|\varepsilon_{sh}-\varepsilon_y|$ is used to separate the problem into the two cases, Fig. 4. If the loop width, $\Delta\varepsilon_s'$, is smaller than this quantity, strain-hardening will be initiated at the same strain value as that under monotonic loading, Fig. 4(b); if the loop is wider, the curve CY will be translated into the appropriate position, Fig. 4(a).

For this hysteretic model, a computer program, BAUSCH, incorporating the above rules, has been written and the results compared with experiments. An example is shown in Fig. 5. Note that all of the hysteretic loops lie to the right of origin. This is characteristic of the time-history for longitudinal bars of a beam subjected to severe cycling moments. During a load reversal, concrete in the compression zone prevents development of high compressive strain in the compressive steel. Therefore, unless concrete crushes and/or spalls off, the development of high compressive strain in reinforcing steel is unlikely. As can be seen from Fig. 5, excellent agreement is found between the experimental and predicted hysteretic loops. Comparisons with other experiments show similar encouraging results.

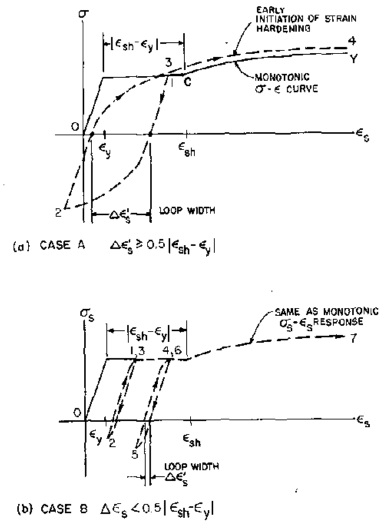


Fig. 4. Strain Hardening for Reversals in Plastic Plateau Range [11].

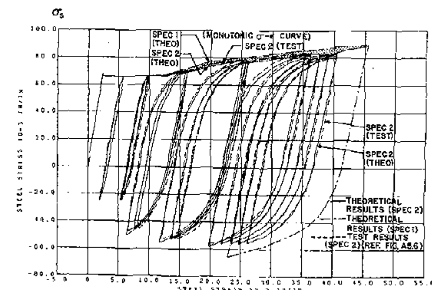


Fig. 5. Comparison of Predicted and Experimental Loops [11].

History Dependent Formulations

Inelastic behavior of steel in cyclic loading is a strongly history dependent or hereditary process. In applying the generalized R-O formulations discussed above, this aspect of the problem is included, but not in a direct manner. In this section two procedures for obtaining cyclic stress-strain relations based on a clearly recognized dependence on the past history of loadings are discussed. One of these has been developed by Kato, Aoki, and Yamanouchi [13,14,15], and the other by Dafalias, Petersson and Popov [16,17].

Kato and his associates demonstrated that if a series of progressively larger hysteretic loops are known for a given material, then, with such data, a monotonic stress-strain curve can be constructed with a reasonable degree of accuracy. In applications the process can be reversed, and cyclic curves can be generated from a conventional stress-strain diagram. In going from a cluster of cyclic hysteretic loops such as shown in Fig. 6 to a monotonic stress-strain curve, only the portions of the curves at stresses of the same sense larger than the ones during the previous cycle are retained. However, to retain the history dependence, the entire curves either above or below the horizontal axis are placed end to end as shown in Fig. 7. For this series of curves the total plastic strain, regardless of its sense, is continuously accumulated along the abscissa.

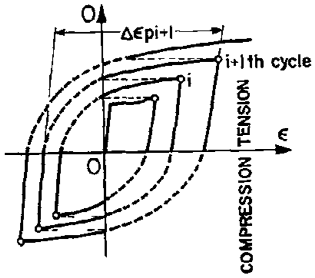


Fig. 6. A Cluster of Hysteretic Loops [13].

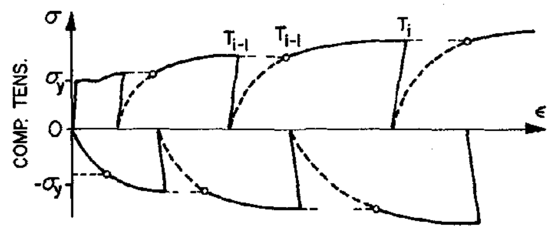


Fig. 7. Arrangement of Hysteretic Loops for Generating a Monotonic Stress-Strain Curve [13].

Inter-connecting the solid lines found in this manner gives an approximate stress-strain curve for monotonic loading. These curves compare very favorably with the ones found from monotonic experiments shown by solid lines in Fig. 8. In generating hysteretic cyclic loops from a conventional stress-strain curve, a suitable function is employed to approximate the downsweep due to the Bauschinger effect. The above history dependent approach of constructing hysteretic loops correctly includes both the effects of isotropic as well as kinematic strain hardening.

Another totally computer oriented approach was developed by Petersson and Popov [17] for determining the plastic behavior for a multi-axial state of stress under generalized loading. However, since this approach reduces to an uniaxial case on which, in fact, it is largely based, it is of direct interest here. To begin with, one can make a general observation that cyclic stress-strain curves tend to approach asymptotically a pair of limiting or bounding lines as shown in Fig. 9 [16]. In dealing with the inelastic or plastic strains it is convenient to remove the elastic part of the strain, resulting in a plot illustrated in Fig. 10. The skewing of the diagram due to the elastic strain component is now removed; but the characteristic bounding lines at a slightly different slope remain intact. This representation is convenient in the discussion which follows.

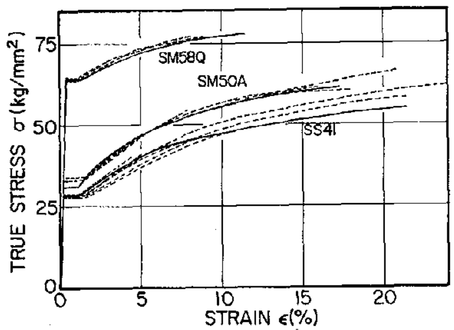


Fig. 8. Comparisons of Predicted and Experimental Curves [15].

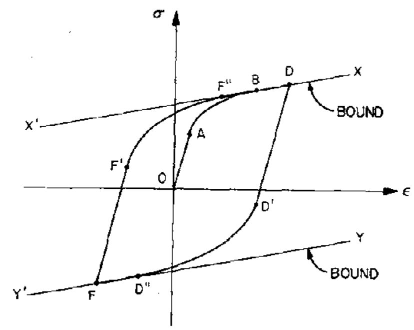


Fig. 9. Schematic Illustration of Line Bounds [16].

Conventional experiments with monotonically applied loading can be used to obtain a stress-plastic strain curve as shown in Fig. 11. The shape of such a curve may be defined with the aid of projections onto the stress axis using sloped bounding lines. The segment AA' on the stress-axis defines the elastic range. Points 1, 2, 3, 1', 2' and 3' for the purposes of illustration only are so chosen as to result in equal stress increments for the different strain increments ε_{p1}, ε_{p2}, ε_{p3}. The yield and bounding circles shown in the figure on the left have no bearing on the uniaxial stress problem being discussed here. Note, however, that the projected distances on the σ-axis between points

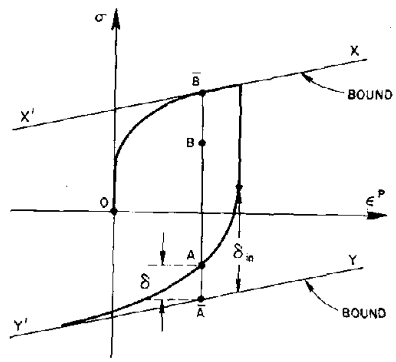


Fig. 10. Line Bounds in Stress-Plastic Strain Space [16].

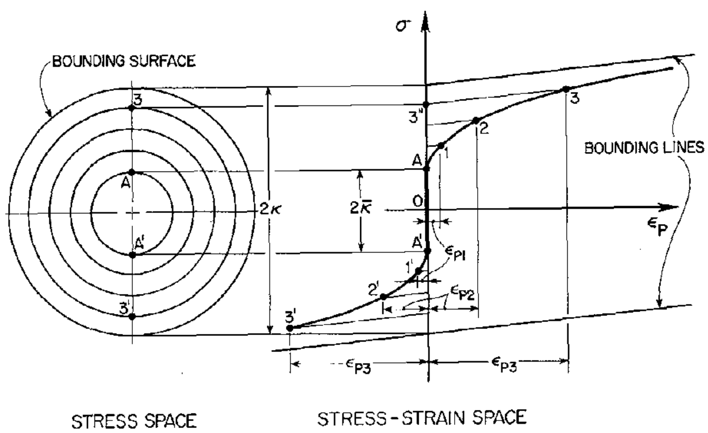


Fig. 11. Typical Stress-Plastic Strain Curves

11', 22', 33', once chosen, remain the same in subsequent work.

Suppose now that at some point B, such as shown in Fig. 12, the stress is reversed and continues to decrease. Then here the segment BC, whose length is equal to the initial segment AA' of Fig. 11, represents the elastic range. If loading were to continue beyond point C, one can make the assumption that for the various plastic strain increments such as ε_{pl}, the vertical segment 11' remains the same as in the virgin curve. Then recognizing that on a load reversal the plastic strain decreases, the correct location of point 1' on the path of deformation is taken at 1". Points 2" and 3" are established similarly. Since this construction makes use of the points B, 1, 2, 3, etc on the previous path of deformation, this becomes a history dependent description of the material behavior. If now the load were again reversed at some point D such as shown in Fig. 13, the curve C3' determines the new locations of points E, 1 (not identified), 2, and 3. In this manner the history dependence of the cyclic process is brought in at each load reversal.

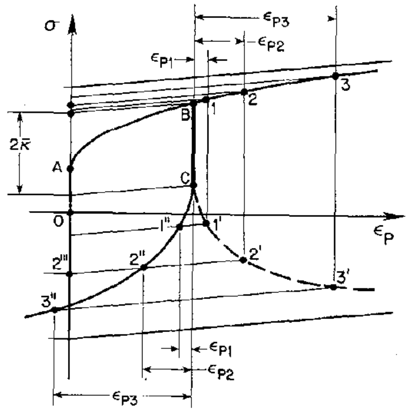


Fig. 12. Definition of Unloading Process

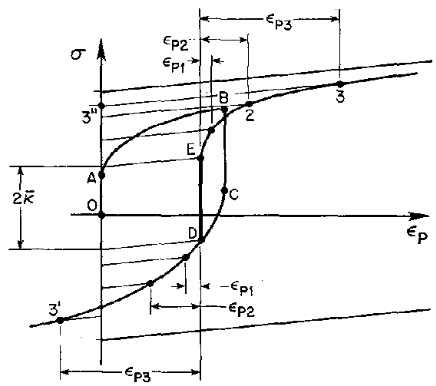


Fig. 13. Definition of a Cyclic Process

Using the above concepts a computer program was developed which requires two further comments. First, the actual representation of the experimental curves is done in a finite element sense using Hermitian polinomials. This scheme is shown in Fig. 14 [17]. Further, there is a good deal of difference in shapes between the initial monotonic stress-strain diagram, and the curve generated during advanced stages of cycling. The proper balance between the two is achieved by means of a weighting function W, Fig. 15. This function depends on the total amount of the accumulated plastic strain up to the time of a load reversal. In the early stages of cycling the initial monotonic stress-strain diagram plays a dominant role. Progressively its influence decreases. Note, however, that the

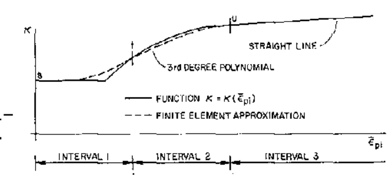


Fig. 14. Finite Element Approximation [17]

abscissae for the lower two diagrams are a function of the plastic strain increments ε_{pi} which correspond to ε_{p1}, ε_{p2}, ε_{p3}, of Figs. 11, 12 and 13. These begin anew at each load reversal. Examples of computer outputs simulating some experimentally determined cyclic curves are shown in Figs. 16 and 17. The agreement between the two sets of curves appears to be rather good.

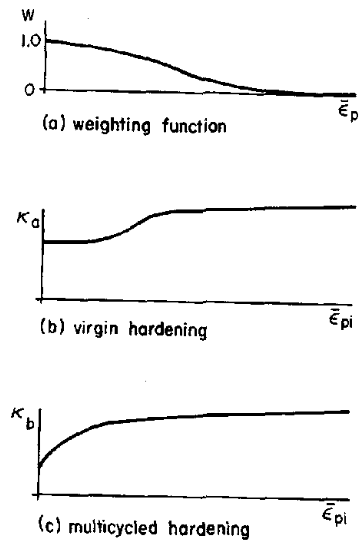


Fig. 15. Weighting Function and Effective Stress-Plastic Strain Curves [17].

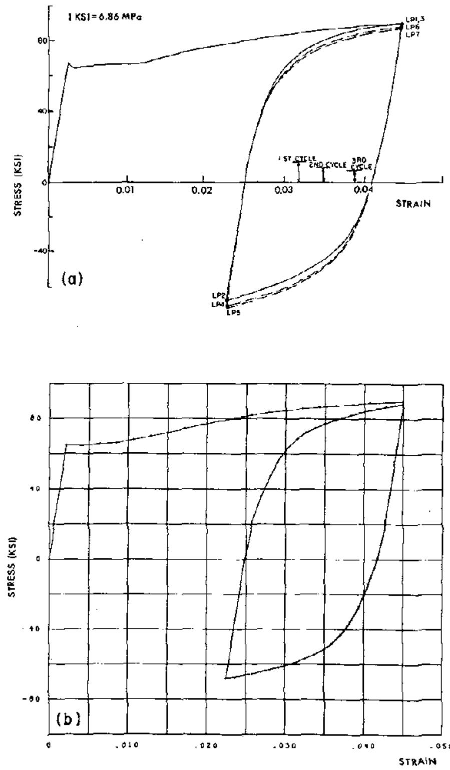


Fig. 16. Comparison of Experimental (a) and Calculated (b) Results [17].

Behavior of Prestressing Steel

Ordinarily prestressed concrete is not used for energy dissipation in aseismic design. Therefore very little research was conducted in this area and there is only very limited information available on cyclic behavior of prestressing steel. The likelihood of prestressing steel for going into the compression range during a seismic disturbance is very remote. Hence the cycling behavior of interest for prestressing steel is the case of progressively increasing strain with an occasional unloading. Precisely this kind of an investigation has been carried out by Blakeley and Park [18]. A typical curve showing stress-strain curves for cyclic loading of a prestressing steel wire is taken from one of their papers, Fig. 18. An examination of this plot shows that the energy dissipation on releasing and reapplying the load is very small. Neglecting this small effect, the conventional monotonic stress-strain diagram can be safely used as an envelope for cyclic loading, provided the steel does not go into compression.

Some Problem Areas

The progress made during the last few years in describing cyclic stress-strain behavior either analytically or with the aid of a computer has been significant. However, some refinements, at least for the benefit of a careful interpretation of experimental results, need further attention. Among these are the following:

1. Ordinarily tensile and compressive properties of steel are assumed to be alike. Such is often not the case, see Fig. 19. Schemes for using an average of the two curves can be readily applied, but may not be sufficiently accurate in interpreting experimental results.

2. In exhibiting the stress-strain data either for monotonic or cyclic loading usually no indication is given regarding the speed of load application. As seismic phenomena are short in duration, more attention may have to be paid to the speed of testing. Perhaps the introduction of the endochronic theory of plasticity into the material interpretation will help.

3. For some refined experiments the agreement between theory and experiments requires further work. The widely accepted assumption introduced by Prandtl that on *releasing the load the material behaves in* an ideally elastic manner, may not be accurate enough. For example, in Fig. 18, corresponding to a strain of 0.030, the stress for the downward path is approximately 67 ksi, whereas for the upward path it is on the order of 107 ksi. In some research investigations this may represent an intolerable inaccuracy.

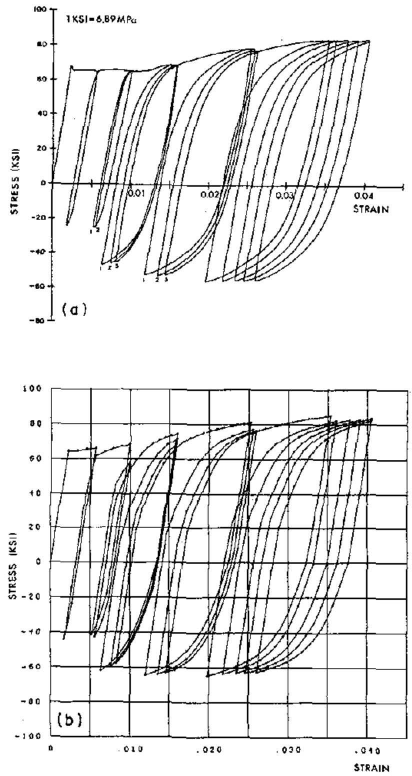


Fig. 17. Comparison of Experimental (a) and Calculated (b) Results [17].

Practical Implications

1. For most practical purposes both the monotonic and the random cyclic stress-strain relations can be sufficiently accurately formulated. In most cases the use of a computer becomes indispensable. All of the available better formulations of the constitutive relations accurately represent the Bauschinger effect. Therefore, at the research level good procedures are available for formulating the constitutive relations. These are impractical in routine design applications, and a need for simpler rules is apparent.

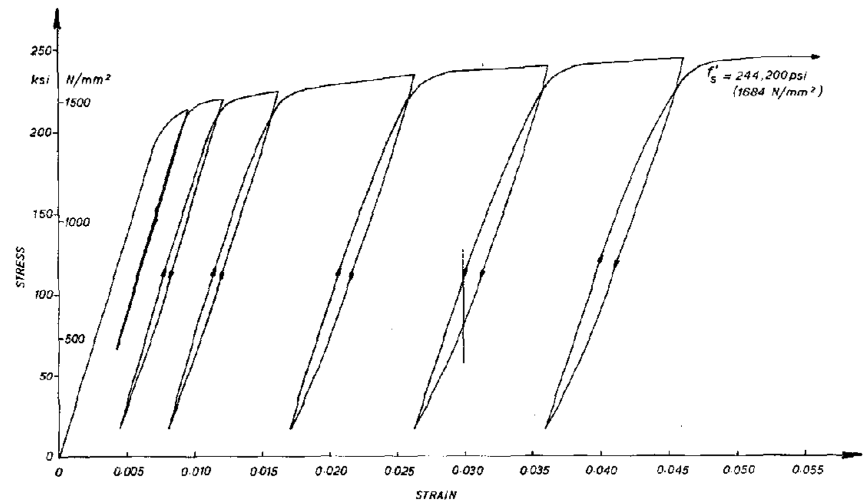


Fig. 18. Stress-Strain Curves for Cyclic Loading of Prestressing Steel [18].

2. Accurate knowledge of the constitutive relations permits a good formulation of the moment-curvature relationship for the couple developed by reinforcing steel in cracked members.

3. A good definition of the reverse curve for the Bauschinger effect provides important information on the tangent modulus of steel, which is necessary for calculating the buckling capacity of the main reinforcement in compression zones of potential cracking which gives a better basis for determining tie and stirrup spacing.

4. The considerable difficulties in accurately defining cyclic stress-strain relations shows that material variability is destined to be very important in aseismic design and the need for non-deterministic approach is indicated.

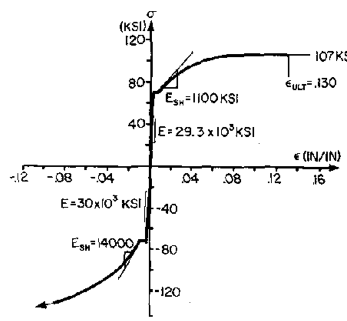


Fig. 19. Stress-Strain Diagram for Reinforcing Steel [3:

BOND AND SLIP OF REINFORCING STEEL

The available information on the behavior of both steel and concrete for monotonic, as well as for cyclic loading, is now sufficiently advanced so as to provide reasonable predictions for their individual behavior. However, their composite action has not been studied thoroughly enough for describing the interaction between the two materials. Thus, although the bond-slip and dowel actions are qualitatively understood at the research level, these phenomena have not been qualified to any degree of generality to be useful in design. Much further work must be done in this area of fundamental importance in reinforced concrete.

Some of the earlier work on bond is ably summarized in 1966 ACI Committee Report, "Bond Stress - The State of the Art." In this document, the nature or bond failure is discussed and the influence of splitting on bond is brought out. However, in terms of the needs for aseismic design, this report does not go far enough. Nothing is said regarding the anchorage and bond behavior under severe repeated and reversed loading conditions. Unfortunately, relatively little has been done in this area since. A brief review of the relevant work will be attempted here. The large number of parameters requiring further study will be enumerated toward the end of this section.

In commenting on bond and slip of reinforcing bars, two levels of approach must be recognized. The one is global; where in a tractable approximate manner the cyclic behavior and its deterioration is described for determining the overall structural response. The second deals in detail with the local phenomena. As information in the first area is virtually unavailable, only comments on the second will be made here.

In general, there are two kinds of bond and slip problems which are encountered in connection with reinforcing bars. First, there is the problem of the interaction between the reinforcing steel and concrete in flexural siutations where cracks perpendicular to the bars can form rather readily. This occurs primarily in slabs, beams and beam-columns. The second problem is one of anchorage, and in the analysis of framed structures, where under extreme conditions deflection ductilities of 6 to 8 may be expected, even a partial pull-out of the bars is very important. Takeda, Sozen and Nielsen [18] were amont the first to note that pull-out of main beam bars from their anchorage causes fixed-end rotations which significantly contribute to the deflection of a frame. In one of their examples, this amounted to over 50% of the deflections caused by cracked beam sections. Ismail and Jirsa [19], and Ma, Bertero, and Popov [11, 20] have found this effect also to be very important. The necessity for including such effects in the analysis seems clear.

Steel-Concrete Interaction in Flexural Members

Two schemes have been used to study the transfer of forces from steel to concrete, and vice versa, around stressed tension bars such as occurs in flexure. In one of these schemes a bar is encased concentrically in a long rectangular prism, or a cylinder, and axial tensile forces are applied to the exposed ends of the bar. The behavior of a bar encased in concrete in a region of a beam in pure flexure is studied in the second scheme.

Injecting ink into tension specimens with concentrically encased bars in long concrete prisms, after splitting them longitudinally, Goto [21] studied the crack pattern. The schematic diagram showing deformation of concrete around reinforcing steel after formation of internal cracks is shown in Fig. 20. This interpretation provides insight into the internal mechanism for monotonic application of force on the bar. One can readily imagine a mirror image of similar cracks that would form if a complete reversal of load were possible, such as would occur in an anchorage under cyclic loading. Such an interpretation under cyclic loading has been used by a number of writers. A large number of similar experiments with monotonically applied end forces was performed by Houde [22], who formulated some bond-slip relations. A refined study with internal instrumentation in the concentric bars for monotonic loading is reported by Nilson [23], who urges further work in this area.

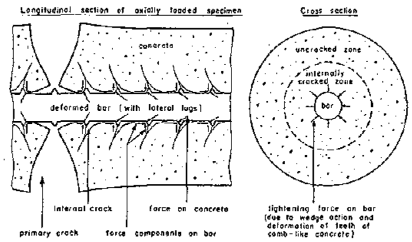


Fig. 20. Deformation of Concrete Around Reinforcing Bars [21]

Valuable as the above studies are, they are not sufficient for resolving bond-slip problems under severe cyclic loading. Using the same experimental set-up as described above, Bresler and Bertero [24] studied the bond problem under cyclic loading. In this simulation it is only possible to apply a tensile force and then release it. However, this may be repeated a number of times with a different level of axial force intensity. Based on these experiments, they reached a number of important conclusions of direct utility in aseismic design. Among these, they noted the history dependence of bond deterioration and the great sensitivity to the maximum peak stress level on subsequent behavior.

A more realistic experimental set-up for studying deterioration of bond in flexural members due to cyclic load reversal than is possible to achieve in the previously referred to experiments was used by Zagajeski [25]. In his arrangement a 12 ft simple span was loaded with reversing third point loads. Therefore, the middle third of the beam was subjected to pure cyclic bending, and it is this region that was studied with regards to the behavior of reinforcing bars in bond. The bars in this region were heavily instrumented, and crack initiators were placed to correlate with the internal gages.

The results of this study led to some significant conclusions. The experiments corraborated the earlier findings that bond effectiveness is sensitive to previous load history. The magnitude of the previous stress level and the sense in which it was acting were found to be important. According to Zagajeski [25], "The essence of bond deterioration lies in the steel-concrete boundary layer. With cycling the tensile concrete boundary layer experiences a softening. The result is that the steel bar encounters less resistance to deformation, and the bond effectiveness deteriorates.... Bond deterioration causes a corresponding reduction in stiffness, as measured by load-deflection and moment-curvature relationships.... With yielding, the

Bauschniger effect in the steel and further cracking contribute to stiffness degradation."

The anchorage of reinforcing steel in concrete is basic to the whole idea of reinforced concrete. Therefore, numerous tests have been conducted on anchorage. However, primarily these were to determine the required length of embedment for developing the full capacity of a reinforcing bar. The amount of pull-out from the anchoring block as a function of the applied force, in the opinion of the writer, did not receive sufficient attention. For gravity load design it may not be too important that bars pull out from their anchoring media, provided the capacity of the bar is developed. However, in aseismic design, at extreme loads, the fixed end rotation caused by the pull-out of the bars may greatly soften the structural system. Therefore, unless this behavior is thoroughly understood and quantified, predictions of structural behavior may be in gross error.

The effect of bar pull-out and push-in is illustrated in Fig. 21 [26], where it is assumed that cracks have been developed on both sides of a column through the whole depth of a beam. In some cases, one must include the effect of concrete contact, if such is re-established. In either case, the end rotation of a member may be very considerable. Recall that in reference 18, it was found that over 50% of a beam's deflection was caused by fixed end rotation.

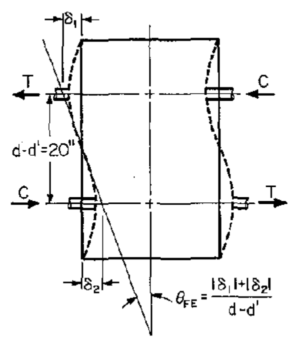


Fig. 21. Fixed-end Rotations at an Interior Column [26]

A brief review of some pertinent work on bar anchorage follows. This will be divided into two parts. First, some information on conventional tests will be given, where the bar is pulled, and in cyclic investigations also pushed, from one side only. Second, a description of some current work on bond will be made where the bar is simultaneously pulled from one side and pushed from the other, such as shown for either the top or bottom bar in Fig. 21. This type of generalized bond test is of particular importance at interior joints.

a) <u>Conventional tests</u>

The prototype of the conventional bond test consists of a concrete block from which an embedded bar is pulled. In some arrangements the block is held in position by a concentric ring support; in others, the block supporting conditions attempt to simulate a part of a beam or a joint, requiring a clamping device or two or more concentrated supports. Reports on a number of tests with monotonically applied, as well as cyclic, loading are available in the literature. Some selections from such work are outlined in the following.

The most recent re-evaluation of test data on development length and splices for monotonic loadings has been made by Orangun, Jirsa, and Breen [27]. In their paper an equation is derived for calculating the development and splice lengths for deformed bars. Emphasis is placed on the amount of cover and spacing of the bars. A more limited aspect on the effect of rib spacings and their heights on bond characteristics of a bar has been reported by Lutz [28].

An extensive study of hooked bar anchorages for monotonic loading is reported in two papers from the University of Texas by Minor and Jirsa [29], and by Marques and Jirsa [30]. In the first paper, primary interest centers on the measurements of slip between the bar and the concrete at several points along the anchored bar. This study shows that the ultimate strength of hooked bar achorages is about the same as that of straight bars, and that 90° hooks are preferable to those of 180°. In the second paper, specimens with hooked bars simulating typical exterior beam-column joints were studied. The degree of concrete confinement at the joint was the principal variable, but these tests, being limited to a single application of the load, must be interpreted with caution for aseismic design.

Some pull-out tests on #8 bars having embedment lengths of 15 in. and 31 in. were carried out by Houde [22]. The specimens were used to study the combination of pull-out and dowel action. Some empirical bond stress-slip relationships are suggested. Before their general acceptance, a wider range of supporting test data is necessary.

Experimental results on cyclic bond behavior began to appear in the 70's. Such work is of direct importance in aseismic design. Some studies of this kind have been conducted in Japan; in the U.S.A. this work is largely concentrated at the Universities of Texas, Washington, and California. One of the better known studies in Japan is that of Morita and Kaku [30, 32]. In their summary paper [31] various load histories of bond deterioration of 19 mm bars in 48 mm embedments , and of 25 mm bars in 66 mm, are presented, and an empirical cyclic bond-slip law is proposed. This work is a good beginning for a rational evaluation of cyclic bond behavior. Their more recent paper [32] discusses cyclic splitting bond failure of large 51 mm (2 in.) deformed reinforcing bars.

In a two paper sequence Brown and Jirsa [33], and Ismail and Jirsa [19] study cyclic anchorage behavior of reinforcing bars in cantilevers. These conditions are typical of exterior joints. In the first paper, the fixed end rotation due to slip in the fixed end is clearly recognized, and some useful experimental data are given. The second paper, specifically directed to the behavior of anchored bars under low cycle overloads, indicates the necessity for increasing the embedment lengths of bars in such cases over and above the current code provisions. These investigators noted that the elongations of the anchored bars contributed between 30% to 45% of the total end deflection of the beams. This conclusion is analogous to that mentioned earlier [18, 20]. The authors conclude [19] that, "...it appears that the response of anchored bars to cycles of large overload is dependent on both the load history and beam geometry, and extensive research will be necessary before valid quantitative results can be obtained...."

Recently some very important work on cyclic pull-out tests has been completed by Hassan and Hawkins [34]. In their experiments concrete blocks simulating the conditions existing at a typical exterior beam-column joint were employed. The blocks were 6 in. thick, 24 in. wide, and 18 in. high with the test bars being placed in the middle of the 6 in. dimension at a distance 6 in. from the top. Two series of experiments were performed with straight #10 Grade 40 bars, and one series, with bars terminating in 180° hooks having 18.4 in. lead-in length. As the test blocks simulated a column, four #7 vertical bars were used in the corners, and four #4 Grade 40 closed ties provided concrete confinement. The blocks were held in position by an appropriate system of supports.

In all cases the bars were well instrumented along their length by placing gages into a machined groove on one side of the bars. Near the pulling end of the bar such gages were placed 2 in. apart. A total of 13 experiments were performed subjecting the specimens to a variety of loading histories. Detailed records of pull-out during load application were maintained.

As a result of these tests some empirical formulae were proposed. These express the energy absorbed, crack lengths, and force-deformation for an anchored bar. The latter attempt is particularly important, but unfortunately as yet the available data are too limited for extrapolation to cases other than those analyzed. A study of the report does, however, shed some light on bond deterioration under cyclic loading, which should help in establishing an acceptable force-deformation relation. The authors of this report state that their "study represents the exploratory part of a major study required to completely identify and solve the problems of extensive damage to buildings as a result of loss of bond in vulnerable beam-column joints." The writer concurs with this opinion.

b) Simultaneous push-pull tests

Work somewhat resembling that of Hassan and Hawkins is being carried out by Viwathanatepa, Bertero and Popov [26], and as their findings as yet are not available in report form, some results are commented upon here in detail. The work is an outgrowth of observing very serious bond deterioration at interior beam-column joints during cyclic loading of subassemblages [20, 35]. In these experiments it was observed that during a severe overload due to lateral forces a crack forms through a beam right next to the column face. On reversing the load, another crack through an adjoining beam forms on the opposite side of the column. If the applied lateral forces are sufficiently intense, cracks form through the whole depth of a beam on both sides of a column. Moreover, at large overloads due to inelastic straining of steel such cracks never close, and the continuous longitudinal bars of the beams are simultaneously pulled from one side and pushed from the other. This condition is shown schematically in Fig. 21. The experimental set-up was designed to simulate this condition for a single bar; and several loading histories for different specimens were investigated. These included a monotonic pull from one side, a monotonic simultaneous pull T from one side and push C from the other with T = C, and several types of cyclic experiments with T = C.

On this project the concrete blocks chosen to simulate a column were 10 in. thick and 46 in. high; the widths of the blocks of the tests considered in this discussion were 25 in. in which #8 Grade 60 bars were placed approximately in the center of the 10 in. width. Eight #7 bars were used as vertical reinforcement, and double #4 Grade 60 closed ties were spaced 4 in. on centers. These overlapping pairs of tiers of approximately 7.5 in. by 15 in. outside dimensions provided excellent confinement of the concrete (compare with the much smaller ties of the Hassan-Hawkins tests). Further, in order not to introduce local stress concentrations, these blocks were held in position by means of heavy prestressed straps running parallel to the test bar at 11 in. on either side of such bars. The design concrete strength was 4000 psi.

Most of the bars were thoroughly instrumented along machined grooves to determine the strains during the progress of a test. In all cases a few cycles at and below the working stress level were performed. The results of the strain measurements for an experiment when the bar was pulled from one side is shown in Fig. 22 [36]. Similar measurements are plotted in Fig. 23 for the later phase of the experiment. From these plots one can clearly sée how the magnitude of the strain increases with increasing bar stress f_s and the nature of strain propagation along the bar.

The stress levels reached in the bars along its length are illustrated in Fig. 24. Note that at the advanced level of loading shown in this figure essentially a constant stress develops near the loaded end of the bar and tapers down to zero at the

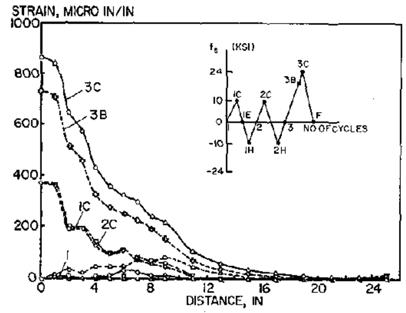


Fig. 22. Strain Distributions Along Bar at Working Loads [36]

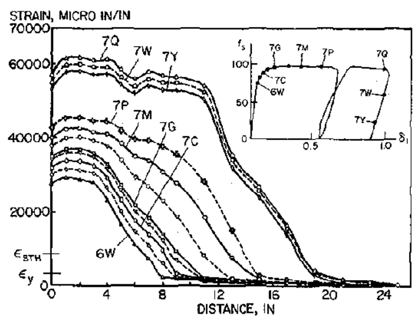


Fig. 23. Strain Distributions at Large Loads [36]

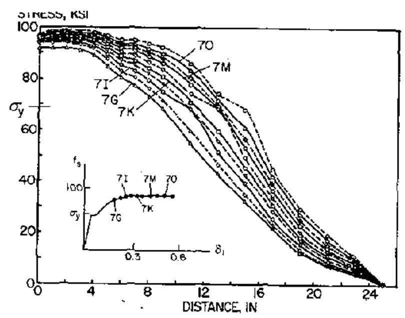


Fig. 24. Stress Distribution along Bar [36]

free end. Figure 25 shows the bond stresses along the same bar. The curves are somewhat jagged, but the general trend is consistent. Some of the undulations are probably due to the difficulties in instrumentation, but others are caused by redistribution of stress due to slip. It is significant to note that at large loads the bond developed near the loaded end is poor. On a gross average the bond stresses reach a magnitude of approximately 1500 ksi. By adding to the external bar movement out of the concrete, the displacements from measured strains, the local displacements along the bar can be found. For this experiment some such results are shown in Fig. 26.

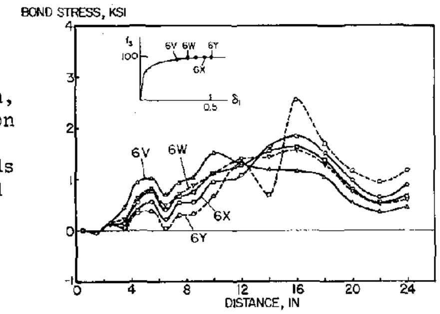


Fig. 25. Bond Stresses Along Bar [36]

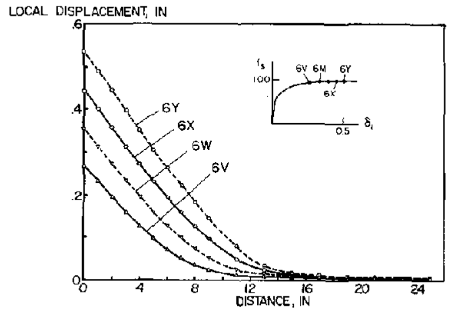


Fig. 26. Local Displacement Along Bar [36]

The results from another experiment in which the bar was pulled from one end and pushed from the other are shown in Figs. 27, 28, 29 and 30. This corresponds to the condition for the top bar in Fig. 21. The left side of the strain distribution shown in Fig. 27 to a horizontally contracted scale resembles that shown in Fig. 23. However, the strain caused by the presence of the compressive force on the right is entirely different. It is clear from the figure that the compressive force C is very rapidly transferred to the concrete. The local displacements of the bar are shown in Fig. 28.

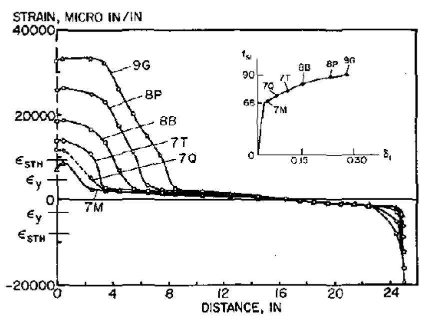


Fig. 27. Strain Distribution Along Bar, T = C [36]

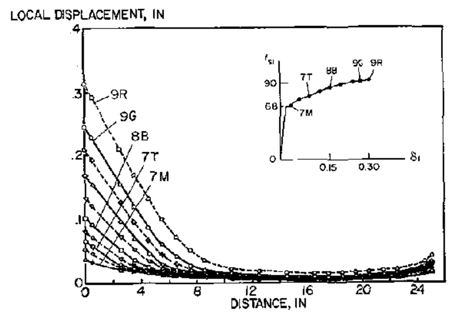


Fig. 28. Local Displacements Along Bar, T = C [36]

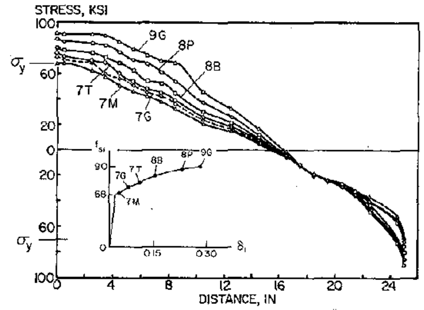


Fig. 29. Stress Distribution Along Bar, T = C [36]

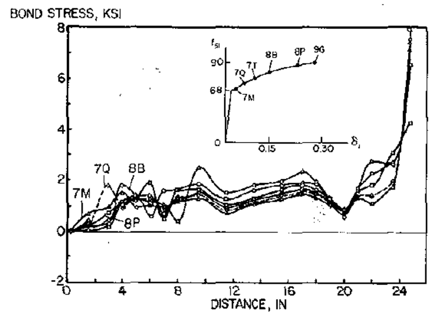


Fig. 30. Bond Stresses Along Bar, T = C [36]

These were determined from external measurements of bar end movements together with the integrated strain along the bar. The stress distribution corresponding to the measured strains is given in Fig. 29, and the bond stresses are shown in Fig. 30. In the latter plot again one can note its jagged character. Same reasons are pointed out in connection with Fig. 25 apply here. Note the huge bond stress developed at the compression end of the bar. At its maximum it reaches 8000 psi, something that was not reported anywhere in the literature.

An example of the results from one of many cyclic experiments [36] are given in Figs. 31-36. In the first figure of this group, Fig. 31, the stresses with T = C against the displacements at the tension end are shown. Note that all of the cyclic curves lie below the monotonic one which is indicated in the figure with dashed lines. Also it is important to note the severely pinched character of the curves at the lower stresses, i.e., considerable displacements of the bar occur even below the working stress level. Figures 32 and 33 show strains along the bar. The irregular pattern of the curves at both ends of Fig. 33 suggests the presence of significant residual stresses as fractured concrete prevents the return of the bar to its original position. The local displacements of the bar are shown in Fig. 34, whereas some selected stresses are illustrated in Fig. 35. Bond stress distribution along the bar can be seen from Fig. 36. A typical value of 1500 psi appears to be applicable for the major part of the bar, except that at the compression end bond stresses on the order of 8000 psi can be noted.

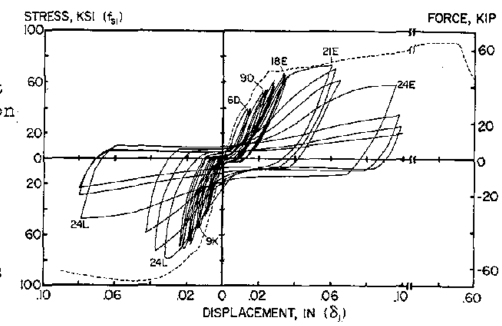


Fig. 31. Cyclic Stress-Displacement Behavior with T = C [36]

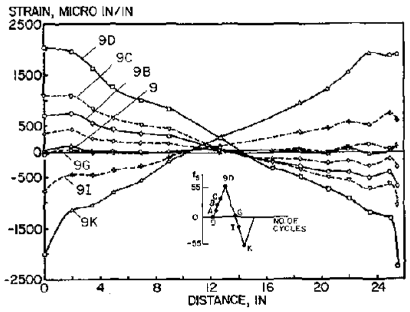


Fig. 32. Strain Along Bar for Cyclic Loading [36]

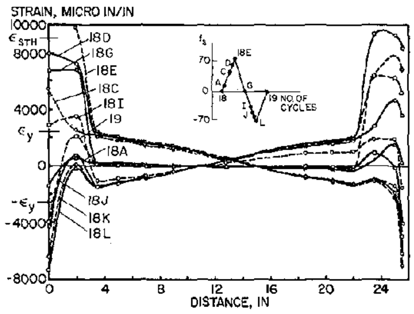


Fig. 33. Strain Along Bar for Cyclic Loading - Advanced Stages of Loading [36]

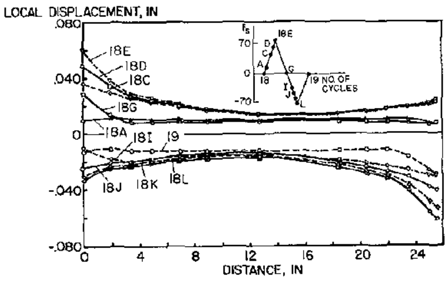


Fig. 34. Local Displacements for Cyclic Loading [36]

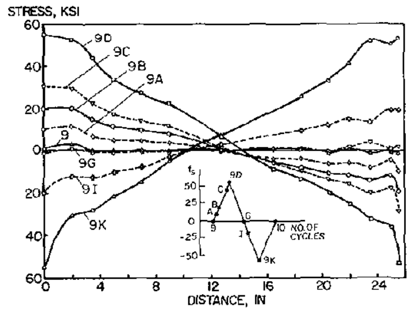


Fig. 35. Stresses Along Bar for Cyclic Loading [36]

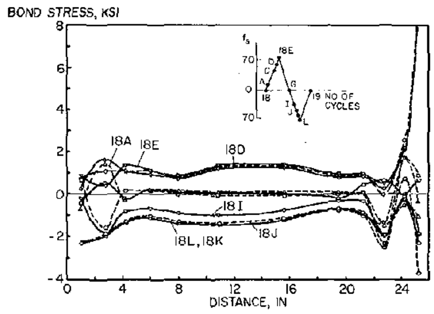


Fig. 36. Bond Stresses for Cyclic Loading [36]

Some Problem Areas

There are extraordinarily many unresolved problems in the general area of bond and slip of reinforcing steel under severe cyclic loading. General rules still await their development. Some of the topics needing immediate attention are noted below.

1. The effect of concrete confinement on bond and the associated slip needs further intensive investigation particularly as it applies to cyclic loading. Only a very few fundamental experiments have been performed in this general area. Some of the subtopics are as follows: a) Bar spacing, concrete cover, etc. appear to be very important. Thus far most of the basic work on cyclic bond was done on single bars. b) The effect on bond of axial compressive forces which are typically present in columns must be studied further. In a pilot test, Cobb [37] has shown the beneficial effect of an axial force. This is illustrated in Fig. 37 where identical specimens with T = C are compared for the cases with and without an axial force. c) Bond experiments in the past have been done at arbitrary loading rates. The effect of loading rates on steel, if not fully known, is at least understood. Nothing has been reported on the rate effect in bond experiments. On the other hand for seismic applications this may be important. d) History of loading must be more carefully scrutinized. Cobb's experiments [37] emphasize the importance of severe initial overloads. Figure 38 shows the consequences of a severe cyclic strain excursion on the subsequent bar slip at working loads.

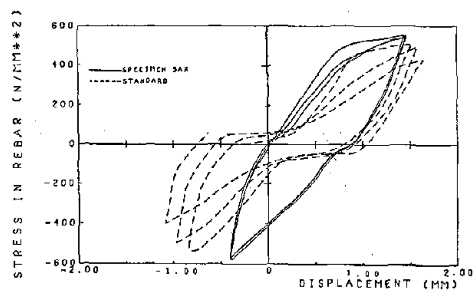


Fig. 37. Stress-Displacement Relationship (Solid Lines for Specimen in Compression) [37]

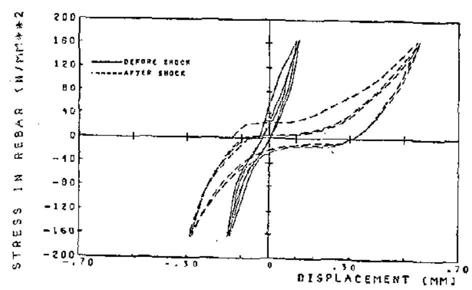


Fig. 38. Stress-Displacement Relationship [37]

2. The bar deformation patterns have evolved without the benefit of cyclic experiments. Changes in the deformation patterns are not too difficult to achieve in practice. Therefore, some effort in studying the optimum bar deformations is desirable.

3. It appears that no one investigated cyclic bond behavior of reinforcing bars in light weight concrete. On the other hand structures made of this material continue to

be built.

4. No systematic information is available on bond behavior of epoxy repaired members. Such information may be in great demand in an aftermath of a major earthquake in a heavily populated area.

5. The interaction of reinforcing steel with prestressing rods in partially prestressed members and the deteriorating effects of dowell action on bond during cyclic loading needs attention.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

CONSTITUTIVE RELATIONS OF STEEL: EFFECTS ON HYSTERETIC BEHAVIOUR OF STRUCTURAL CONCRETE MEMBERS AND ON STRENGTH CONSIDERATIONS IN SEISMIC DESIGN

by

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INTRODUCTION

Concrete has little use as a structural material without the addition of steel in the form of reinforccing bars or prestressing tendons which, when properly detailed, give it strength and ductility. Knowledge of the stress-strain characteristics of steel under monotonic, repeated, and cyclic (reversed) loading is required if the behaviour of structural concrete members subjected to general forms of loading is to be determined. This paper reviews the stress-strain properties of steel and discusses the influence of these properties on the moment-curvature relationships of structural concrete members. The need for account to be taken of probable steel overstrength in seismic design is emphasized.

STRESS-STRAIN BEHAVIOUR OF REINFORCING BARS

Monotonic Stress-Strain Behaviour

Typical stress-strain curves for steel bars used in reinforced concrete obtained from monotonic load tests are shown in Fig. 1. Generally the stress-strain curves found from tension and compression tests are sufficiently similar to be assumed to be identical. When the yield point is reached the curves generally exhibit a yield plateau in which the strain increases with little or no stress increase until strain hardening commences. Various idealizations for the monotonic stress-strain curve have been proposed. The ACI Code [1] in strength design assumes an elastic-perfectly plastic relationship which ignores the strength increase due to strain hardening. If more accuracy is required, the strain hardening region of the stress-strain curve can be represented by a rising straight line, or by a curved line, obtained by fitting curves to experimental stress-strain data. Equations representing the strain hardening region of the stress-strain curves, based on limited test data, are available (see for example [2]).

The length of the yield plateau is generally a function of the strength of the steel. High strength high carbon steels generally have a much shorter yield plateau than low strength low carbon steels. Similarly, the cold working of steel can cause the shortening of the yield plateau to the extent that strain hardening commences immediately after the onset of yielding. High strength steels also have a smaller elongation before fracture than low strength steels.

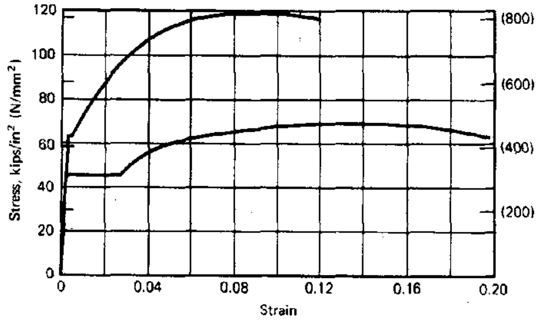


Fig. 1 Typical Stress-Strain Curves for Steel Reinforcing Bars

It is essential for the safety of the structure that the steel be ductile enough to undergo large plastic strains before fracture. Steel specifications generally specify elongations at fracture which are reasonable for seismic design.

A rapid rate of loading will increase the yield strength of steel. For example, for steel with static yield strength f_y = 45, 51 and 57 ksi (310, 352 and 393 MPa, respectively) the following yield strength increases due to strain rate have been reported [3]:

Average strain rate in/in per second	0.001	0.01	0.1	1.0
Percentage increase in yield strength	2 to 5%	7 to 14%	16 to 21%	25 to 28%

Repeated Stress-Strain Behaviour

If the load on a steel specimen is released before failure, recovery occurs along a stress-strain path that is parallel to the initial elastic branch. If loaded again the same stress-strain path will be followed up to the original curve as in Fig. 2, with perhaps a small hysteresis and/or strain hardening effect. The monotonic stress-strain curve is then followed as if unloading had not occurred. Hence repetitions of loading of the same sign can be idealized by sets of lines parallel to the original elastic branch, and the monotonic stress-strain curve gives a good idealization for the envelope curve for the repeated loading case.

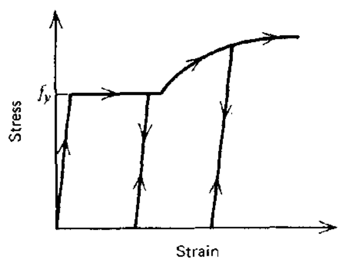


Fig. 2 Stress-Strain Curve for Steel Under Repeated Loading

Cyclic Stress-Strain Behaviour

A stress-strain curve for steel under cyclic (reversed) loading is shown in Fig.3. After the first yield excursion the loading curve becomes nonlinear at low stresses due to the Bauschinger effect. This steel behaviour is strongly influenced by previous strain history.

Kent and Park [4], Thompson [5], Aktan, Karlsson and Sozen [6], Ma, Bertero and Popov [7], and others, have produced analytical methods based on the Ramberg-Osgood equation with empirical constants to trace the cyclic stress-strain curve.
A typical form of the Ramberg-Osgood equation is

$$\varepsilon_s - \varepsilon_o = \frac{f_s - f_o}{E_s}\left(1 + \left|\frac{f_s - f_o}{f_{ch} - f_o}\right|^{r-1}\right) \qquad (1)$$

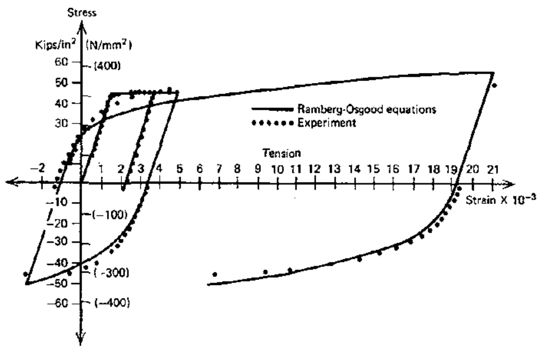


Fig. 3 Stress-Strain Curve for Steel With Cyclic Loading [4]

where ε_s and f_s are the strain and stress on the curve, E_s is the modulus of elasticity in the initial elastic loading run, ε_o and f_o are the strain and stress at the beginning of the curve, and f_{ch} and r are empirical parameters.

In the method of Kent and Park [4] the unloading branches of the stress-strain curve for stresses of both signs are assumed to follow the initial elastic slope,but the loading portions of the curve after the first yield excursion are given by Eq. 1. The parameter f_{ch} was found to be a function of the yield strength and the plastic strain produced in the previous loading run, and r was found to be a function of the loading run number. Fig. 3 compares an experimental stress-strain curve with the curve given by Eq. 1 using the empirical values found for f_{ch} and r [4].

The monotonic stress-strain curve with origin at the initial position has been shown by Leslie [8], and others, to approximately describe the envelope curve for cyclic loading provided cyclic loading occurs in the tensile strain region or in the compressive strain region. For more symmetrical tension - compression straining the envelope origin appears to be displaced horizontally. A closer fit with experimental stress-strain curves than the model proposed by Kent and Park has been obtained by Thompson [5] who assumed that the envelope curve for cyclic loading is the same as the monotonic loading envelope except that whereas the origin of the stress-strain curve for monotonic loading is at (o,o), the origins for cyclic loading curves are assumed to be at (ε_{zmx},o) for tensile loading and (ε_{zmn},o) for compressive loading, where ε_{zmx} and ε_{zmn} are the tensile and compressive residual strains, respectively, *from the previous load run if the recovery* curve is linear, as illustrated in Fig. 4. The stress is not allowed to exceed the envelope curve during loading. The full extent of the curve is represented by Eq. 1, i.e. the unloading branch is assumed to be nonlinear. The parameter f_{ch} was found to be a function of the yield strength and the plastic strain produced in the previous cycle, and r was found to be a function of the plastic strain produced in the previous load cycle. The empirical values for f_{ch} and r found for Grade 40 steel (f_y = 276 MPa) are *given elsewhere* [5].

Aktan, Karlsson and Sozen [6] have also used a form of the Ramberg-Osgood equation to define both the loading and unloading branches of the curve and obtained good agreement with test results, as is shown in Fig. 5 where the curve R-O refers to the Ramberg-Osgood model. They also devised

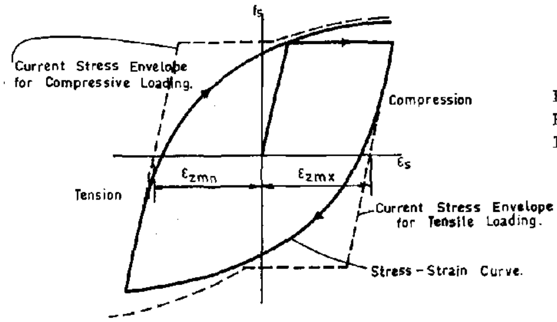


Fig. 4 Stress-Strain Envelope Proposed for Cyclic Loading of Steel [5]

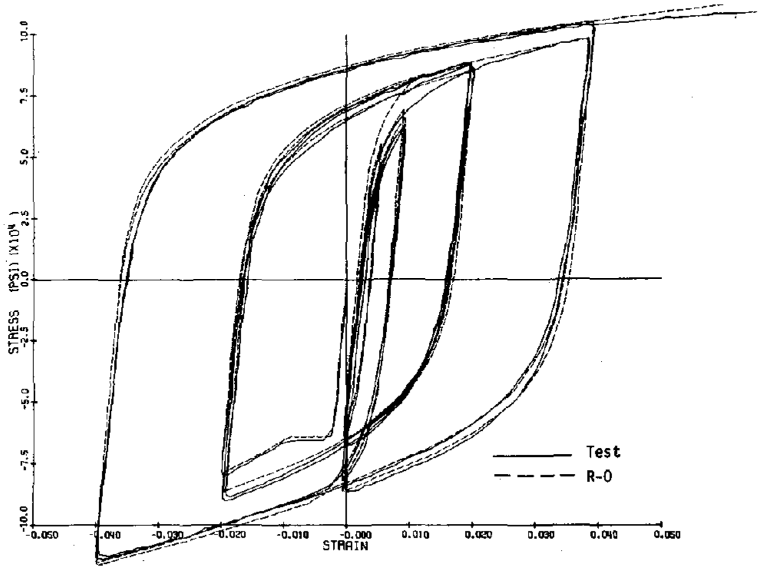
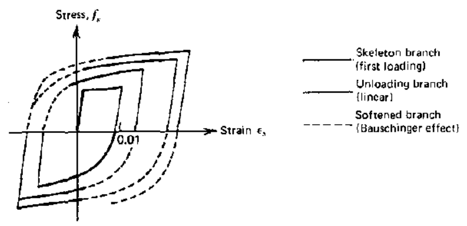


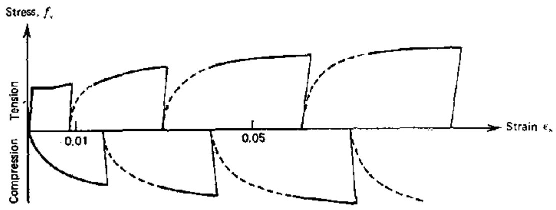
Fig. 5 Stress-Strain Curve for Steel With Cyclic Loading [6]

an alternative idealization consisting of sets of straight lines parallel to the elastic slope and inclined to it. Ma, Bertero and Popov [7] have also obtained close fit with experimental stress-strain data using a form of the Ramberg-Osgood equation.

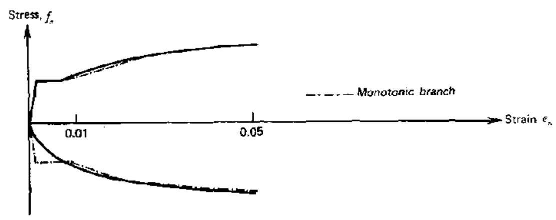
An alternative type of idealization by Kato, Akiyama and Yamanouchi [9], based on the observation of experimental stress-strain data, derives the stress-strain relationship for cyclic loading from the monotonic curves for tension and compression in the manner illustrated in Fig. 6. The measured cyclic stress-strain curve (Fig. 6a) is divided into curves corresponding to loadings attained for the first time, unloading branches (straight lines), and



(a) Cyclic load curves



(b) Expanded curves



(c) Skeleton curves

Fig. 6 Stress-Strain Curves for Steel With Cyclic Loading [9]

loadings attained in previous cycles (softened curves due to the Bauschinger effect). The parts of the diagram of the same sign of stress can be plotted in sequence, as in Fig. 6b. Connecting the segments of the first loading branches end for end (Fig. 6c) leads to a diagram similar to the monotonic curves. A difference exists at the initial part of the curve in compression, which is considerably curved, compared with the monotonic curve. Kato, et al, represented the softened curves, due to the Bauschinger effect, by hyperbolas commencing at zero stress. Using this idealization the cyclic stress-strain curves can be obtained approximately from the monotonic curves.

The main aspect which needs to be clarified by further tests is how the monotonic stress-strain curve should be used as an envelope, if at all. For example, whether the assumption of Kato, et al, illustrated in Fig. 6 is more accurate than the assumption of Thompson illustrated in Fig. 4 for tension - compression straining. This has an important effect on the level of stress reached and the influence of previous load cycles on the strain at fracture. However there seems little doubt that for cyclic loading in either the tensile strain region or the compressive strain region the monotonic stress-strain curve with origin at the initial position gives a satisfactory envelope curve.

MOMENT-CURVATURE BEHAVIOUR OF REINFORCED CONCRETE

The strength and ductility of a plastic hinge region of a member is dependent on the moment-curvature relationship, which in turn is based on the stress-strain relationships for the steel and concrete as well as on the steel areas and concrete dimensions.

Monotonic Moment-Curvature Behaviour

Fig. 7a shows a doubly reinforced concrete beam section. The stress-strain curve for the steel shown in Fig. 7b, measured from some samples of high strength steel, has strain hardening commencing at four times the yield strain, and a tensile strength of 1.7 times the yield strength reached at a strain of 0.12. The concrete stress-strain curve shown in Fig. 7c is based on test data which takes the confining effect of the stirrup ties in the

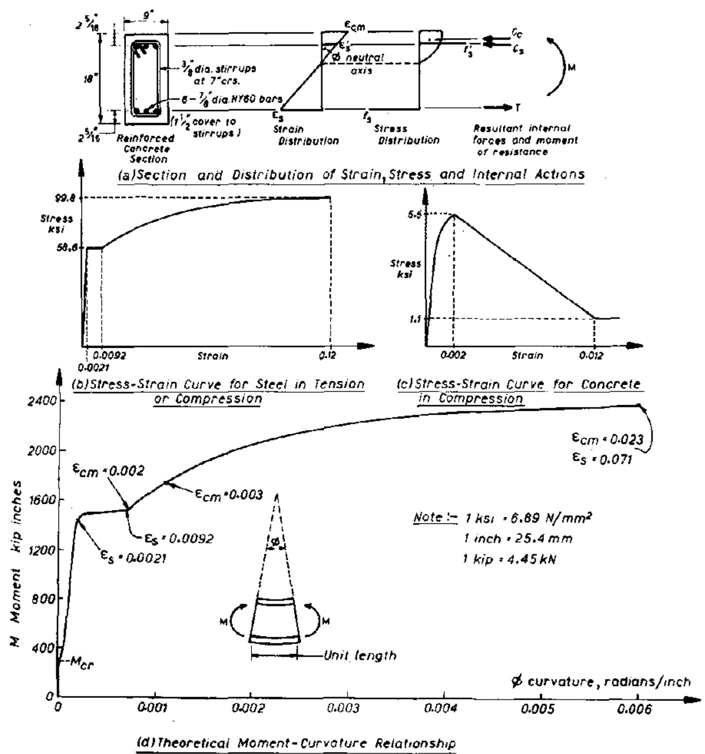


Fig. 7 Reinforced Concrete Beam and Monotonic Moment-Curvature Relationship

member into account [2]. The theoretical moment-curvature curve shown in Fig. 7d for the section was calculated using the assumed stress-strain curves, assuming that plane sections remain plane, and satisfying the requirements of strain compatibility and equilibrium to determine the variation of moment with curvature. The strains in the tension steel (ε_s) and at the extreme compression fibre of the concrete (ε_{cm}) are shown on the curve of Fig. 7d. It is evident that strain hardening of the steel has caused a considerable increase in the moment capacity at high curvatures. For example the moment at a curvature of 15 times the yield curvature is 45% greater than the flexural capacity calculated ignoring strain hardening.

Repeated Moment-Curvature Behaviour

Since the monotonic stress-strain curve for steel forms the envelope curve for repeated stress of the same sign, it is evident that the monotonic moment-curvature relationship will also give a good approximation for the envelope curve for repeated moment of the same sign.

Cyclic Moment-Curvature Behaviour

Theory for the moment-curvature characteristics of reinforced concrete sections subjected to cyclic flexure can be derived using the relationships for the stress-strain curves for steel and concrete under cyclic loading. Plane sections are assumed to remain plane and an iterative technique which ensures compatibility of strains and equilibrium of forces at the section may be used. Fig. 8 shows the theoretical moment-curvature curve calculated for a doubly reinforced concrete beam section compared with the measured experimental points [10]. For the beam reinforcing steel f_y = 47 to 48 ksi (324 to 331 MPa) and strain hardening commenced at 16 to 18 times the yield strain.

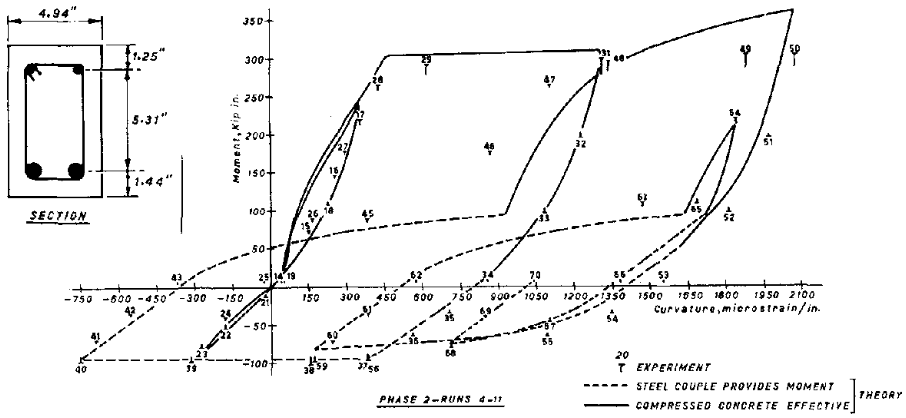


Fig. 8 Moment-Curvature Relationships for a Doubly Reinforced Concrete Beam Section With ρ = 3.54%, ρ' = 1.14% and ρ_s = 2.30% Subjected to Cyclic Loading [10] (1 kip in = 113 Nm, 1 in = 25.4 mm)

The theoretical curves were calculated between the experimental curvature points at which reversal of flexure took place. The theoretical and experimental curves compare reasonably well. It was found that the shape of the stress-strain curve for steel has a large influence on the behaviour of reinforced concrete sections subjected to intense cyclic flexure,because for large portions of the moment curvature loop after first yield of the steel open flexural cracks exist in the concrete over the whole cross section of the member and the moment of resistance is provided only by the forces in the reinforcing steel. The dashed part of the theoretical curve in Fig. 8 shows the regions where full depth cracking exists. The open cracks exist in the compression zone because of the plastic elongation of the steel in the previous loading run which leaves open cracks which only close when the steel yields in compression. The experimental points in Fig. 8 do not illustrate such a sudden change of stiffness due to closing of the cracks in the concrete as the theoretical curves, since in practice some compression will be transferred across cracks before they completely close due to shear displacements across cracks and the presence of loose particles in the cracks. Fig. 8 also illustrates the rounding of the moment-curvature loops due to the Bauschinger effect of the steel. Another influence of the Bauschinger effect on members subjected to cyclic loading is to reduce the tangent modulus of the steel at low levels of stress, and this could lead to buckling of the reinforcing bars in compression at lower levels of load than would be expected from monotonic loading tests.

For unsymmetrically reinforced sections the small area of steel (in the top of the beam section of Fig. 8) when yielding in tension will not have sufficient tensile force capacity to cause the large area of steel (in the bottom of the beam section of Fig. 8) to yield in compression. Thus the large area of steel will only be subjected to tensile yielding and the cracking caused by it will never close. The small area of steel, however, will yield in both tension and compression. For symmetrically reinforced members, the compression steel will only yield when the tension steel yields causing the crack in the compression zone to close near the end of each loading run. However the concrete in the compression zone will prevent the development of extremely high compressive strains in the steel, whereas when the steel is strained in tension extremely high tensile strains can develop, as is evident from the neutral axis position in the section. Therefore the cyclic stress-strain loops of the longitudinal reinforcing bars are likely to remain primarily in the tensile strain range. Hence the type of test load cycling illustrated in Fig. 3 is more representative of conditions under cyclic loading in a beam than that illustrated in Figs. 5 and 6.

Previously it was indicated that a rapid rate of monotonic loading increased the yield strength of steel. Cyclic loading tests conducted by Mahin and Bertero [11], on reinforced concrete beams subjected to high and low strain rates, indicated an increase in the first yield moment of about 20% due to high strain rate, but a reduction in the effect of high strain rate occurred at greater deformations, and after the first cycle of loading in which the member is yielded the hysteresis loops were little affected by the strain rate. Thus there is good justification for ignoring the effect of high strain rates on the material strengths in seismic design.

STRESS-STRAIN BEHAVIOUR OF PRESTRESSING STEEL

Prestressing steel is unlikely to be stressed far into the compression range during seismic loading and hence cyclic loading of prestressed concrete members will cause repeated loading to occur primarily in the tension range. Fig. 9 shows a stress-strain curve for a prestressing steel wire measured during repeated load tests. The monotonic load curve forms an accurate envelope curve for the repeated load stress-strain hysteresis loops. The envelope curve can be conveniently idealized as two straight lines connected

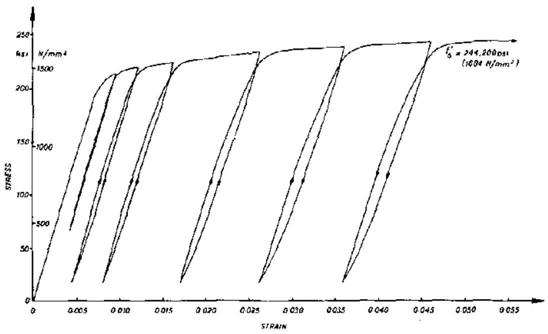


Fig. 9 Stress-Strain Curves for Prestressing Steel With Cyclic Loading [12]

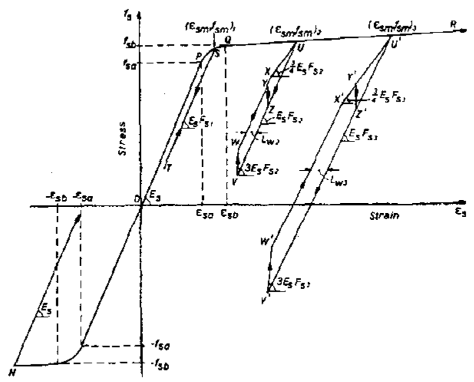


Fig. 10 Idealized Stress-Strain Curves for Prestressing Steel With Cyclic Loading [12]

by a hyperbolic curve, and idealizations for the repeated stress-strain behaviour taking hysteresis into account can be devised. An idealization due to Blakeley and Park [12] is shown in Fig. 10. Thompson [5] has recently produced a more accurate idealization which uses the Ramberg-Osgood equation to more accurately follow the hysteresis loops.

MOMENT-CURVATURE BEHAVIOUR OF PRESTRESSED CONCRETE

Theoretical moment-curvature curves for cyclic loading of prestressed concrete sections can be computed using cyclic stress-strain curves for steel and concrete [12,5]. A comparison of theoretical and experimental moment-curvature relationships is shown in Fig. 11 for the plastic hinge section of a prestressed concrete beam [12]. Preliminary trials during the development of the theory showed that it was necessary to include the effect of hysteresis in the cyclic loading idealization for prestressing steel in order to obtain good agreement between the theoretical and experimental curves. The moment-curvature loops illustrate the well known characteristic of prestressed concrete, that there is little energy dissipation due to hysteretic damping prior to the commencement of crushing of concrete in compression, since the initial tension in the tendons results in considerable deflection recovery of the member on unloading.

The stress-strain characteristics of prestressing steel, and the range of steel stresses during cyclic loading of the member, indicate that the monotonic moment-curvature relationship is, with good accuracy, the envelope curve for cyclic loading. This observation is confirmed by examining cyclic moment-curvature relationships, such as in Fig. 11.

At the computed flexural strength of the member, using conventional theory which assumes an extreme fibre concrete compressive strain of 0.003,

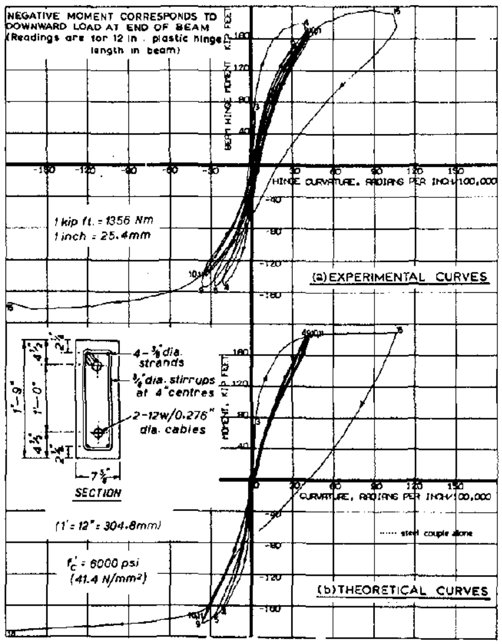


Fig. 11 Moment-Curvature Relationships for a Prestressed Concrete Beam Section Subjected to Cyclic Loading [12]

the tensile strain in the prestressing steel in the tension zone will generally be greater than 0.01. Thus at high curvatures the steel stress will approach the tensile strength, particularly when the neutral axis depth of the section is small.

DESIGN ASPECTS CONCERNING STRENGTH

Consequences of Overstrength

If during a severe earthquake the longitudinal steel at the plastic hinges in the beams of a frame reaches a stress which is significantly greater than the strength used in design, the resulting increase in the flexural capacity of the beams will have the following undesirable consequences: (1) The increase in moment capacity of the beams will increase the bending moments acting on the columns and may cause plastic hinges to form in the columns, possibly resulting in a brittle column sidesway collapse mechanism, (2) the increase in moment capacity of the beams will result in an accompanying increase in shear forces acting on the beams and the columns that could lead to a brittle shear failure, (3) the increase in steel strength could cause a shear failure of the beam column joint cores, and (4) the increase in steel strength could cause an anchorage failure. Thus, overstrength of flexural steel in beams could lead to undesirable behaviour unless the designer is prepared to increase the flexural strength of the columns, the shear strength of the beams, columns, and joint cores, and the anchorage lengths. These comments emphasize that in seismic design there is a danger in using overstrong steel. In the design for gravity loads overstrength is not of great importance.

There is a school of thought that has recommended the use of strain hardening steel in seismic design on the grounds that the increase in moment capacity results in less structural damage during a severe earthquake. While that may be so, the consequences of overstrong steel need to be considered carefully, as discussed above.

Reinforced Concrete Structures

The previous considerations have shown that the plastic hinge behaviour of reinforced concrete members is very dependent on the stress-strain characteristics of the steel. Therefore, the method for determining the envelope stress-strain curves from the monotonic stress-strain curves illustrated in Figs. 4 and 6 should also apply to determining the envelope moment-curvature relationships from the monotonic moment-curvature relationships. However, it has been commented that most of the cyclic loading in the reinforcing steel will occur in the tensile range of strain. Therefore the monotonic moment-curvature relationships with the origin at the original position should describe the envelope curves for cyclic flexure with good accuracy, and should give a good indication of the flexural strength levels liable to be reached during severe seismic loading.

At a plastic hinge section in a beam during severe earthquake, strains in the tension steel in the order of 10 to 20 times the strain in the steel at first yield, or higher, may be reached. The tensile steel strain reached may be particularly high when several excursions into the yield range occur during the earthquake. If the steel has a short yield plateau and strain hardening occurs soon after yielding, the steel may reach a stress considerably higher than the yield strength during severe seismic ground shaking. The large increase in flexural strength of members reinforced by Grade 60 (f_y = 414 MPa) steel due to strain hardening, illustrated in Fig. 7, makes it essential that such stress increase be taken into account in the design of seismic resistant structures. For Grade 40 (f_y = 276 MPa) steel, strain hardening does not occur so early (see Fig. 1), but nevertheless the strength increase may be significant.

In order to avoid the consequences of overstrength it is suggested that the design shear forces in beams and beam-column joint cores, and the input moments into columns, should be calculated on the basis of a steel stress at the beam plastic hinges of αf_y, where f_y is the specified yield strength of the steel and α is an overstrength factor which is greater than unity. In this suggested procedure the beams are designed for flexure using a steel strength f_y, and when the longitudinal beam steel has been allocated the other design actions are calculated on the basis of that beam steel acting at stress αf_y. The overstrength factor α can be derived considering the sum of two effects: (a) the amount the probable yield strength exceeds the specified yield strength, and (b) the amount the steel stress at the anticipated maximum strain exceeds the probable yield strength due to strain hardening. It is evident that the determination of α for a particular steel can only be carried out on the basis of statistical data for the steel properties.

For New Zealand produced reinforcing bar the determination of α is made difficult by the scatter of measured yield strengths which show a trend of increasing strength with decreasing bar diameter, and a scatter of strains at which strain hardening commences. For example, for steel with f_y = 40 ksi = 275 MPa strain hardening typically commences at a strain of 9 to 18 times the yield strain, and for steel with f_y = 55 ksi = 380 MPa strain hardening typically commences at 2 to 4 times the yield strain. It is probable that considerable scatter occurs for steel in any country. The recommendation made in New Zealand is α = 1.25 for steel with f_y = 40 ksi = 275 MPa and α = 1.40 for steel with f_y = 55 ksi = 380 MPa. These values appear to arise from a yield strength increase of 15% plus a strain hardening increase of 10% for steel with f_y = 40 ksi = 275 MPa, and a yield strength increase of 6% plus a strain hardening increase of 34% for steel with f_y = 55 ksi = 380 MPa, at strains of at least 10 times the yield strain. However quoting such percentages gives a false idea of accuracy since the overstrength factors are rounded approximate values.

It is evident that to ensure that the strength increase due to strain hardening is kept to within known bounds, steel specifications should place an upper limit on the ratio of tensile strength to yield strength. Also, efforts should be made to encourage steel producers to increase the length of the yield plateau of high strength steel.

The possible buckling of compression steel during cyclic loading is also of concern due to the reduction of the tangent modulus of the steel at low stress levels resulting from the Bauschinger effect. With this in mind it is suggested that the spacing of transverse reinforcement providing lateral support to longitudinal bars should not exceed six longitudinal bar diameters. This spacing requirement has been recommended in New Zealand.

Prestressed Concrete Structures

At the ultimate moment capacity of ductile prestressed concrete members the stress in the prestressing tendons in the tension zone will already be close to the tensile strength. To avoid the consequences of overstrength it is suggested that when calculating the design shear forces in beams and beam-column joint cores, and the input moment into columns, the prestressing steel stress in the tension zones of plastic hinge regions of beams with bonded tendons should be taken as the probable tensile strength of the steel.

It is also important that the steel be adequately ductile. It is suggested that the strain at fracture of prestressing steel should be not less than 4%.

CONCLUSIONS

The stress-strain characteristics of reinforcing steel and prestressing steel subjected to monotonic, repeated, and cyclic loading can be idealized with reasonable accuracy. However the envelope curve for cyclic loading of reinforcing steel needs clarification, and additional test data is necessary to develop more general expressions for use in the idealizations.

The moment-curvature characteristics of reinforced and prestressed concrete members are strongly dependent on the steel stress-strain characteristics. The inelastic compressive strains induced in the steel will be much smaller than the inelastic tensile strains in the steel during cyclic loading and the monotonic moment-curvature relationship could be used to obtain the envelope moment-curvature relationship.

The attainment of high steel strengths at plastic hinge zones in beams, due to steel strengths greater than specified and to strain hardening, could enforce brittle modes of collapse of a structure unless the enhanced steel strength is taken into account when calculating the design shear forces and column actions. In New Zealand it is recommended that when calculating those design actions the beam steel stress should be taken as $1.25f_y$ for steel with f_y = 40 ksi = 275 MPa and $1.40f_y$ for steel with f_y = 55 ksi = 380 MPa, where f_y is the specified steel strength. For prestressing tendons the steel stress at plastic hinges in beams can be taken as the probable tensile strength.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

DEVELOPMENT LENGTH REQUIREMENTS FOR REINFORCING BARS UNDER SEISMIC CONDITIONS

by

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University of Washington

INTRODUCTION

In his paper "Mechanical Characteristics and Performance of Reinforcing Steel Under Seismic Conditions," McDermott has thoughtfully examined requirements for steels but with the apparent implicit assumption that it is primarily the stress-strain characteristics of the steel that determine the deformation characteristics of hinging regions. Where that hinging region is adjacent to a joint, as in the case of the beam-column subassemblages tested by the PCA, the bond characteristics for the reinforcement are at least as important as its stress-strain characteristics, for determining the deformation characteristics for the hinging region [1]. That result has been clearly established from simulated beam-column and slab-column tests conducted at the University of Washington over the last five years. Since the importance of bond in determining the moment-rotation characteristics of slab-column connections can be only indirectly established, this discussion concentrates on the findings from the simulated beam-column tests.

Tests have been made on over 40 simulated beam-column connections [2, 3, 4] about one third of the specimens having the proportions shown in Fig. 1a and the other two-thirds the proportions shown in Fig. 1b. The specimens modeled the joint region of an exterior beam-column subassemblage. Beam moments alternating in direction, were simulated by pushing or pulling on the beam bar and reacting that force by restraints R_3 for tensile moments and R_4 for compressive loadings. Column moments and axial forces were simulated by restraints R_1 for tensile loading and R_2 for compressive loading.

Variables examined have been as follows: (1) load history, monotonic loading to failure, cyclic zero to a maximum loading to failure, partially reversed cyclic (maximum compressive load on beam bar significantly less than maximum tensile load), and fully reversed cyclic (maximum displacement for compressive loading equal to that for tensile loading); (2) bar grade, 40 or 60; (3) bar size, No. 6 or No. 10; (4) concrete strength, 2600 to 5100 psi; (5) hooks on the end of the bar, 90° or 180°; (6) bar deformations, bamboo style or alternating V-style and (7) the amount of hoop reinfercement in the simulated joint. Specimens were proportioned, so that the loaded beam bar would yield before failure and so that the amount of hoop reinforcement was at least double that at which the attack end slip became sensitive to hoop reinforcement for monotonic loading tests to failure.

The principal test results have been as follows:

(1) The form of the specimen, Fig. 1a or 1b, has had little effect on the load-attack end slip curve when essentially identical specimens of the two types have been tested.

(2) The characteristics of the loading history had a marked effect on the rate of bond deterioration and mode of failure. Two modes of failure have been observed (a) collapse following attainment of the same ultimate load and deformation capacities as those obtained for a similar specimen loaded monotonically to failure, and (b) collapse due to bond deterioration at an ultimate load considerably less and at an ultimate deformation about one-third of that for a similar specimen loaded monotonically to failure. Collapse in the later mode occurred when the bar was reversed cyclically loaded to yielding both in tension and compression. If, however, reversed cyclic loading was discontinued before failure and the specimen then loaded monotonically to failure the failure mode reverted to type (a).

(3) For reversed cyclic loading specimens with V-type lugs always performed better than those with bamboo type deformations. Typically for cycling between the same constant peak displacements the number of cycles for failure for V-type lugs was about double that for bamboc type deformations.

(4) For reversed cyclically loaded hooked bars, bond resistance and energy absorption are provided initially by the "lead-in" length to the hook and there is a change in behavior once slip penetrates to the end of that "lead-in" length. Shown in Fig. 2(a) is the load-slip response for a straight bar and in Fig. 2(b) the response for the same bar terminated with 180 degree hook. The broken curve indicates the response for a monotonically loaded specimen. The response with 180 degree hooks is much poorer than with straight bars because once slip penetrates to the hook the motions of the hook break up the connection. Shown in Fig. 3(a) is the load slip response for a straight bar and in Fig. 3(b) the response for the same bar terminated in a 90 degree hook. The hooked connection maintains good characteristics for tensile loading considerably longer than for compressive loading but even then its characteristics are not nearly as good as those for a specimen with a straight bar. An additional advantage of a 90 degree hook over an 180 degree hook is that for tensile loadings to displacements beyond the displacement for the peak capacity, there is some regaining of strength with increasing tensile displacements.

(5) The grade of the bar has less effect than the general form of its stress-strain characteristics. The slope of the load-slip curve after yielding depends on the length of the yield plateau in the bar's stress-strain curve and the bar's strain hardening modulus. For bars with similar strain hardening moduli the slope of the post-yield load slip curve decreases as the length of the yield plateau increases and for bars with similar yield plateau lengths the slope of the post-yield load slip curve increases as the strain-hardening modulus increases. Thus the total response is an averaging of two effects. That behavior reflects the manner in which stresses build up along the bar. Strain measurements showed that anchorage lengths of only 10 bar diameters were needed to develop yielding in a grade 60 bar and about 8 bar diameters for a grade 40 bar. Thus, when a bar is first stressed inelastically any yielding length is small and the initial slope of the post-yield load-slip curve depends primarily on the strain-hardening modulus. However, the bond stress that can be developed with a yielding bar is considerably less than that with an elastic bar. Therefore, for increasing loads beyond yielding, the length of bar that is yielding increases rapidly and the length of the yield plateau becomes increasingly important in determining the slope of the load-slip curve.

(6) For specimens of the same geometry subjected to the same load history the slip for maximum load was approximately inversely proportional to the yield strength of the bar and directly proportional to the concrete compressive strength.

(7) Before slip penetrated to the end of the bar or the lead-in length for a bar with a hook, the load-slip curve were spindle shaped and stable for cycling between constant slip limits. The load-slip curves directly reflected the cyclic stress-strain curve for the reinforcing steel. After slip developed at the end of the bar on the end of the lead-in length the load-slip curves became characteristically S-shaped and for cycling between constant slip limits, the capacity and stiffness decreased rapidly with increasing cycles. For a straight bar, or a bar terminating in an 180 degree hook the capacity always decreased as the slip increased beyond that for bond failure. For a bar terminated with 90 degree hook, there was a gain in capacity with increasing displacements beyond that for slip at the end of the lead-in length. However, that gain was lost rapidly with cycling. Further, for compression loadings there was no gain and the bars behaved as if they were straight.

(8) The maximum capacity and the displacement for that capacity for reversed cyclic loading were insensitive to increases in amounts of hoop reinforcement in the joint above that shown in Fig. 1(b).

From these studies it is apparent that the load-slip characteristics for the bar are as important as its stress-strain characteristics for determinations of the response of hinging regions in seismically loaded structures. In Fig. 4 a comparison is made between the measured moment-rotation responses of beam-column subassemblages II and V tested by the PCA (5), and the responses predicted from the University of Washington's load-slip tests when all the rotation of the beam with respect to the column is assumed to be caused by slip of the reinforcement within the joint. The moment-rotation relationships are those for the first major inelastic cycle applied to the specimens. Subassemblage II had hoop reinforcement in the joint more than adequate to prevent any marked effect on the specimen's response of diagonal cracking or bulging within the joint core. Except for a complete absence of joint reinforcement specimen V was identical to specimen II. For specimen II the theoretical and measured moment-rotation relationships are in close agreement. For specimen V the agreement is much poorer. That result is to be expected since the load-slip data used was that for specimens with "adequate" hoop reinforcement and the post-yield load-slip characteristics deteriorate rapidly as the hoop reinforcement is reduced below the "adequate" level.

The formulas recommended in Chapter 12 of ACI 318-71 for development lengths are not adequate for inelastically, reversed cyclically loaded reinforcing bars. The formulas recommended by ACI-ASCE Committee 352(6) should be used provided additional embedment length is added to recognize that the concrete beyond the line of the column reinforcement at the loaded end of the bar is ineffective for bond.

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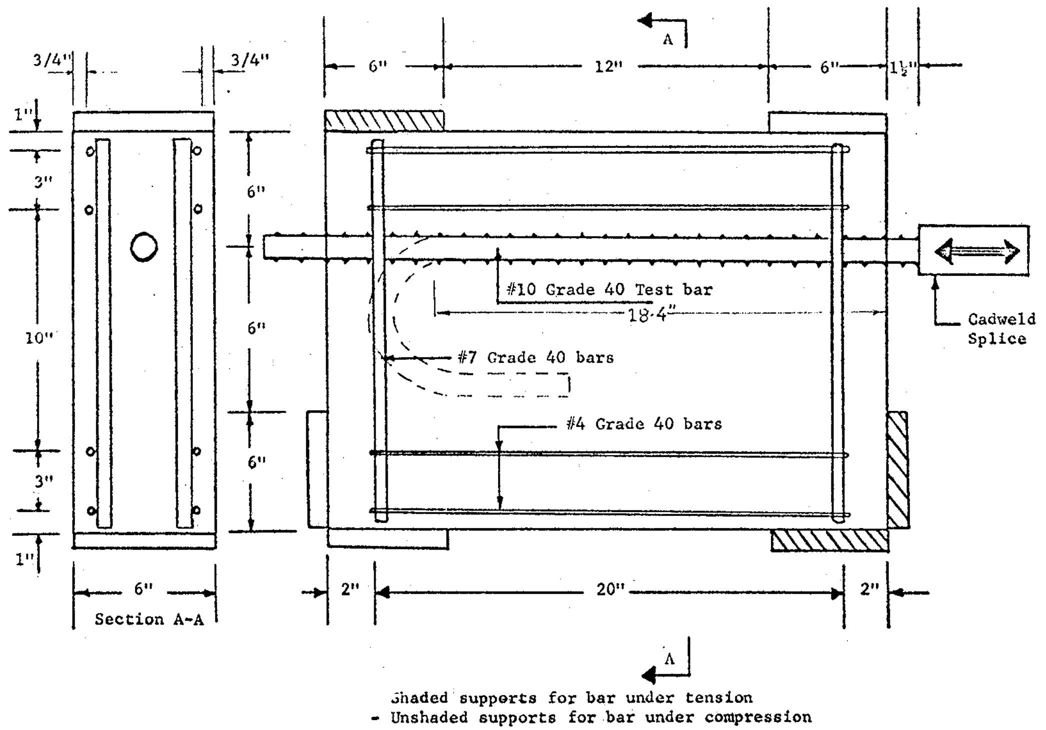
RECOMMENDATION

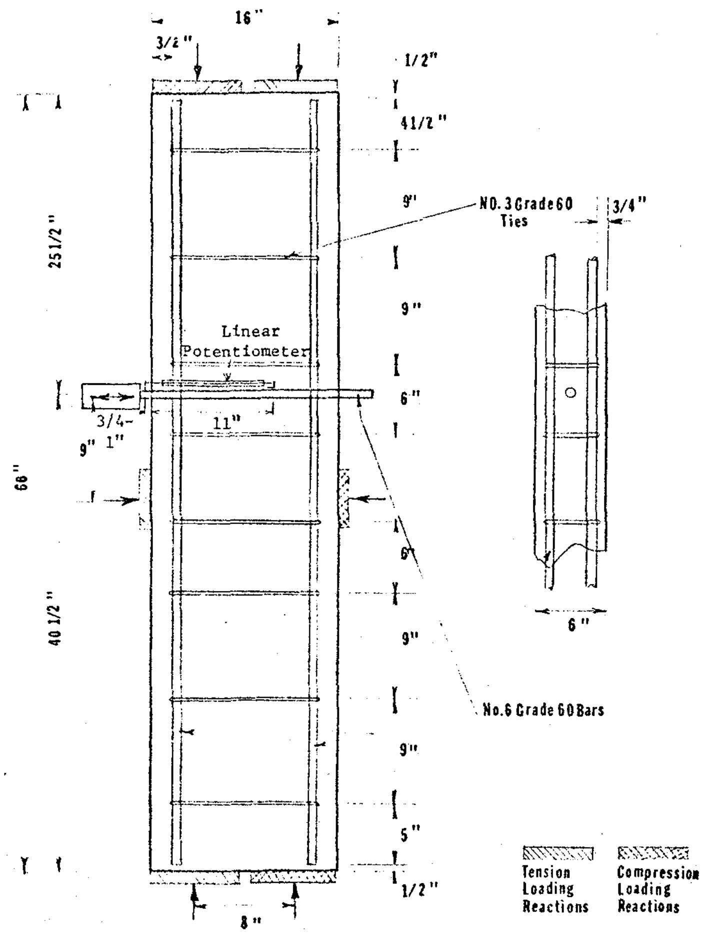
CONDUCT A COMPREHENSIVE INVESTIGATION TO DETERMINE THE FACTORS AFFECTING THE LOAD-SLIP RELATIONSHIPS FOR BARS CYCLICALLY LOADED INTO THE INELASTIC RANGE

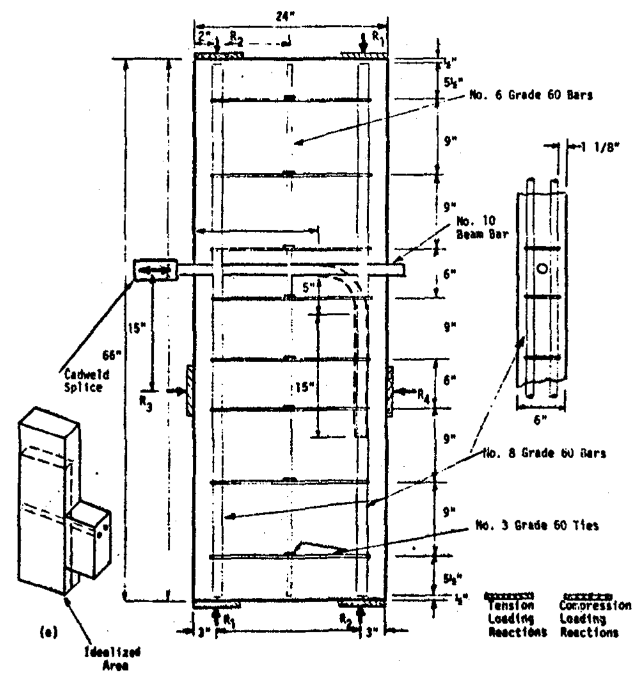
The load-slip relationships for bars anchored within connection regions markedly affects the rotational characteristics of adjacent hinging regions. In addition, the provisions of ACI 318-71 for development length are not adequate for structures that may be subjected to severe cyclic loading. A comprehensive set of experiments should be made and expressions developed that predict load-slip relationships for a variety of geometric and reinforcement conditions for bars cyclically loaded into their yield range. From those results, recommendations should also be made for development length provisions for bars located in structures in seismic zones.



(a) Reference 2

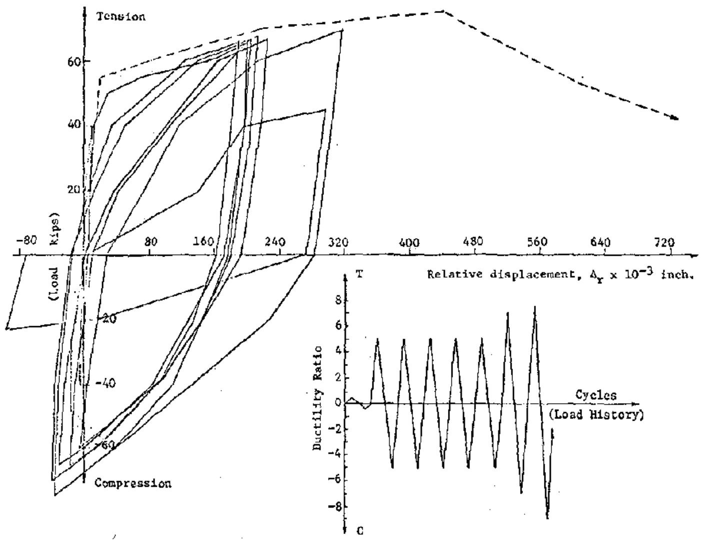
FIG. 1 TYPICAL TEST SPECIMENS



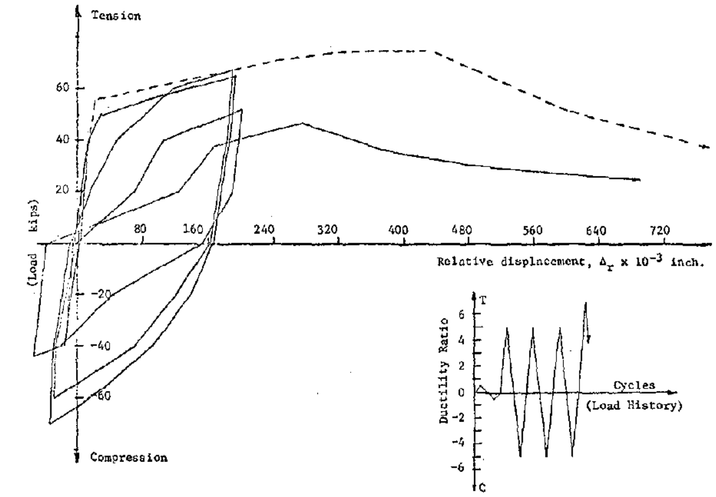


(b) References 3 and 5

FIG. 1 (Cont'd): TYPICAL TEST SPECIMENS



(a) Straight Bar



(b) Bar Terminated with 180° Hook

FIG. 2 LOAD-SLIP CURVES FOR NO. 10 BARS, GRADE 40
3200 PSI CONCRETE, V-TYPE DEFORMATIONS

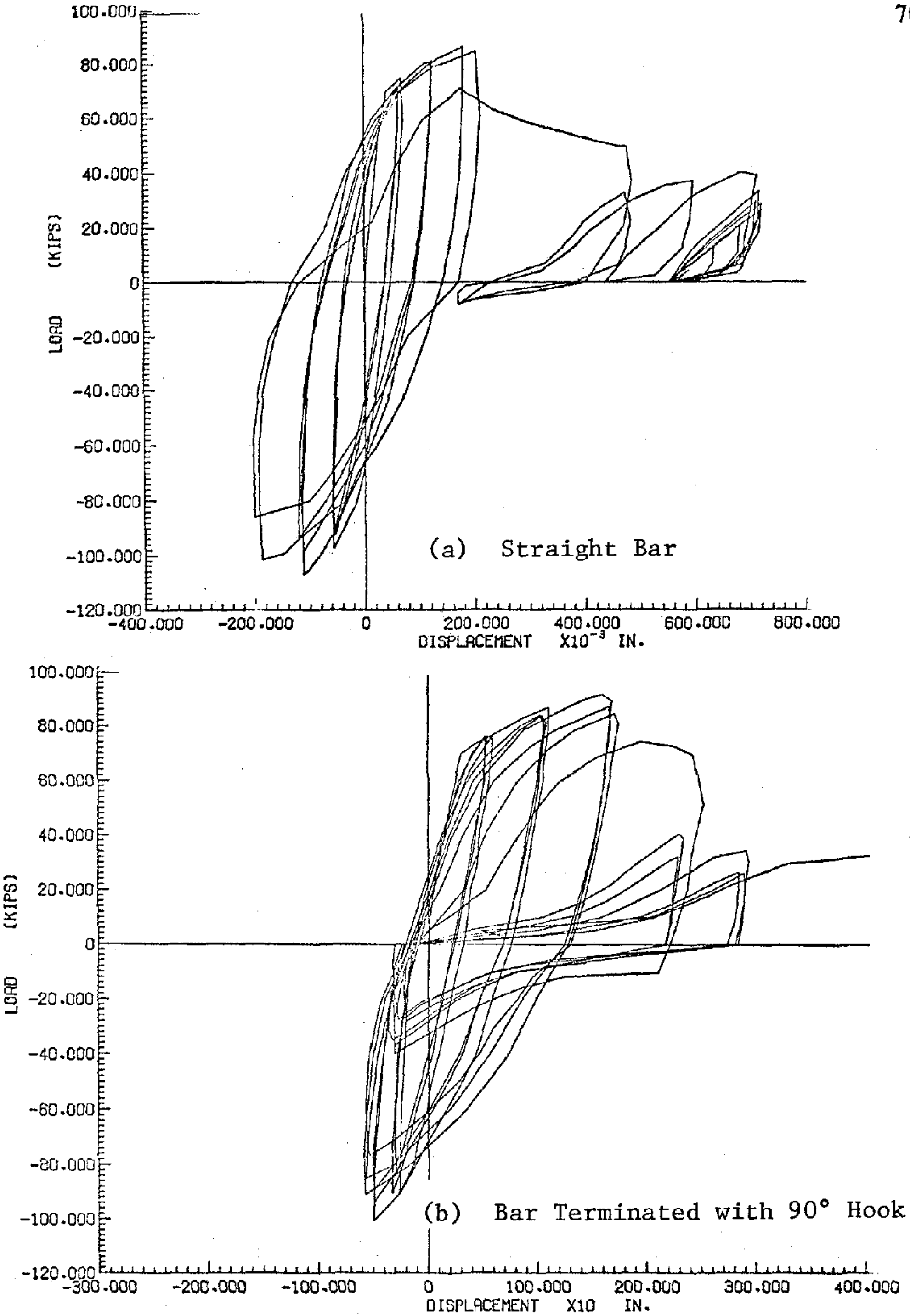


FIG. 3 LOAD-SLIP CURVES FOR NO. 10 BARS, GRADE 60
4-100 PSI CONCRETE, BAMBOO DEFORMATIONS

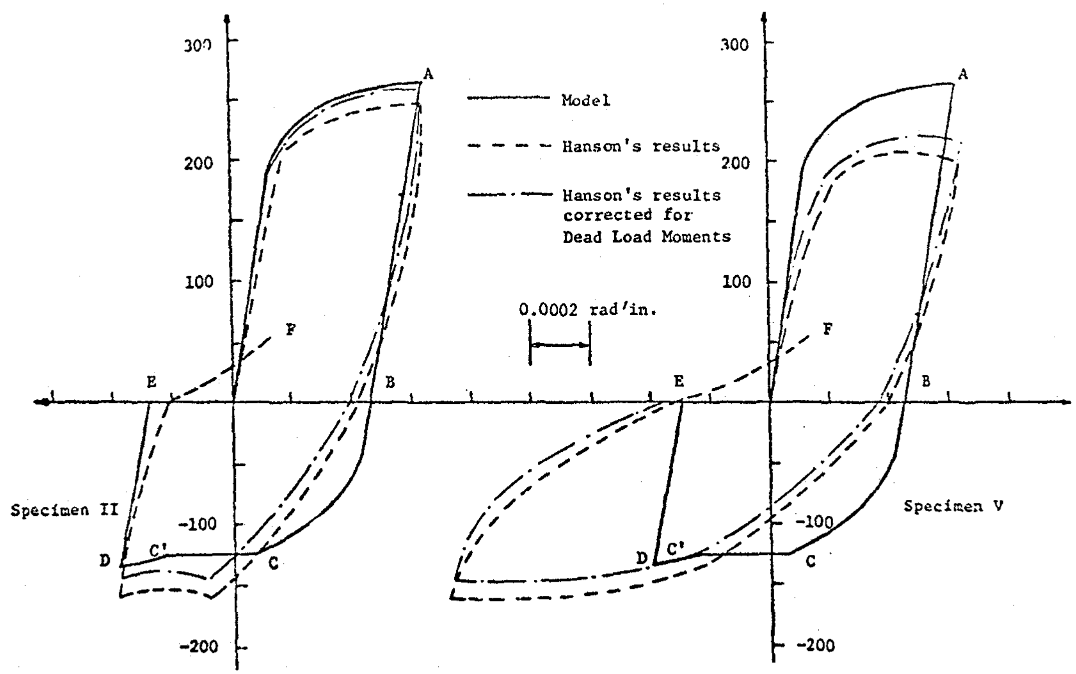


FIG. 4 MEASURED AND PREDICTED MOMENT-ROTATION RELATIONSHIPS FOR PCA BEAM-COLUMN SPECIMENS

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REINFORCED AND PRESTRESSED CONCRETE STRUCTURAL SYSTEMS, INCLUDING TYPES OF FOUNDATIONS: IMPORTANCE OF CONCEPTUAL DESIGN

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED
CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

STRUCTURAL SYSTEMS FOR EARTHQUAKE RESISTANT CONCRETE BUILDINGS

by

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INTRODUCTION

The vertical load-resisting capability of a building is its reason for existence. However, with increasing building height, the lateral loads due to wind, earthquakes, etc. assume more and more importance. This is because, with increasing height, the overturning effect of such loads increases. Also, with increasing slenderness, the lateral displacements and interstory displacements may endanger overall structural stability and the integrity of nonstructural elements, and may cause discomfort to occupants. The challenge to the structural engineer in designing a multistory structural system lies in providing the necessary stiffness against lateral loads in a way which will require the least premium for height over the cost of supporting the gravity loads. Structural engineers have met this challenge by developing efficient, economical and innovative new structural systems for buildings ranging in height to over 100 stories.

This paper reviews the reinforced concrete structural systems that have evolved over the last few decades. Resistance to wind was the prime consideration in their development, since, until relatively recently, tall buildings were mostly built in nonseismic areas. This report focuses on the seismic resistance of these structural systems. An important distinction must be drawn here between forces due to wind and those produced by earthquakes. These loads are sometimes thought to belong to the same category, just because codes specify both in terms of equivalent static forces. Although both wind and earthquake loads are dynamic in character, a basic difference exists in the manner in which they are induced in a structure. Whereas wind loads are external loads applied, and hence proportional, to the exposed surface of a structure, earthquake loads are essentially inertial forces. The latter result from the distortion produced by both the earthquake motions and the inertial resistance of the structure. Their magnitude is thus a function of the mass of the structure, rather than its exposed surface. Also, in contrast to structural response to essentially static gravity loading or even to wind loads, which can often be validly treated as static loads, the dynamic character of the response to earthquake excitation can seldom be ignored.

The lateral load resisting reinforced concrete structural systems are described here in general terms, before converging on the seismic resistance of such systems.

LATERAL LOAD RESISTING REINFORCED CONCRETE SYSTEMS

The three basic framing systems to resist lateral loads in high-rise concrete buildings are: (1) frames, (2) structural (shear) walls coupled or acting individually, and (3) frames interacting with structural walls.

Reinforced concrete frame structures depend mainly on the rigidity of member connections for their resistance to lateral forces, and generally tend to be uneconomical (require high premium for height) beyond 20 stories. This is particularly true of the conventional frame structure consisting of two sets of mutually perpendicular frames. Recourse usually has to be taken to other systems for buildings taller than 15 to 20 stories high [1].

The introduction of deep vertical elements or structural walls represents a structurally efficient solution to the problem of stiffening a frame system. This is illustrated in Fig. 1a. A structural wall behaves essentially as a vertical cantilever beam, while a frame exhibits the deformations typical of a shear beam under transverse loads. The interaction between the two elements reduces the lateral deflection of the structural wall at the top, while the wall helps support the frame near the base. However, except perhaps where the walls are located along the exterior of a building or form the elevator shaft, some degree of architectural flexibility may have to be sacrificed with their use. In many cases, a judicious disposition of walls in plan allows them to function efficiently as vertical and lateral load resisting elements without interfering much with architectural requirements. Fig. 1b shows typical plan arrangements of frame-wall systems. Reinforced concrete structures using systems similar to these have been built to a height of 70 stories [1].

Utilization of shear panels--structural walls, one or a few stories in height--scattered throughout the plan, and shifted in location to offer architectural flexibility while supplying sufficient rigidity, is another way to adapt to diverse architectural and functional requirements [2].

The staggered wall-beam system is an innovation suitable for residential buildings. Although only a limited number of high-rise buildings have been built using this system, its advantages of large unobstructed areas in the typical floor, of column-free areas under the buildings for parking, and of high rigidity in the transverse direction may eventually lead to broader applications [2].

A modification of the conventional frame arrangement which has been found economically suitable for buildings up to about 60 stories high is the so-called "framed tube". In this structure, a typical plan of which is shown in Fig. 1c, the exterior columns in what would otherwise be a conventional frame are spaced more closely together and are connected by relatively deep spandrel beams to form an exterior grid which is usually designed to resist the bulk of the lateral load. The framed tube represents a logical evolution of the conventional frame structure, possessing the necessary lateral stiffness with excellent torsional resistance, while retaining the planning flexibility which isolated interior columns allow [1,3].

For taller structures, an arrangement which has been found particularly suitable for office buildings is the so-called "tube-in-tube" system [1,3]. A typical plan is shown in Fig. 1d. This system has emerged as a logical solution to the problem of providing a tall, stiff structure with wide column-free spaces between a central core which houses all services and an external peripheral grid of closely spaced elements.

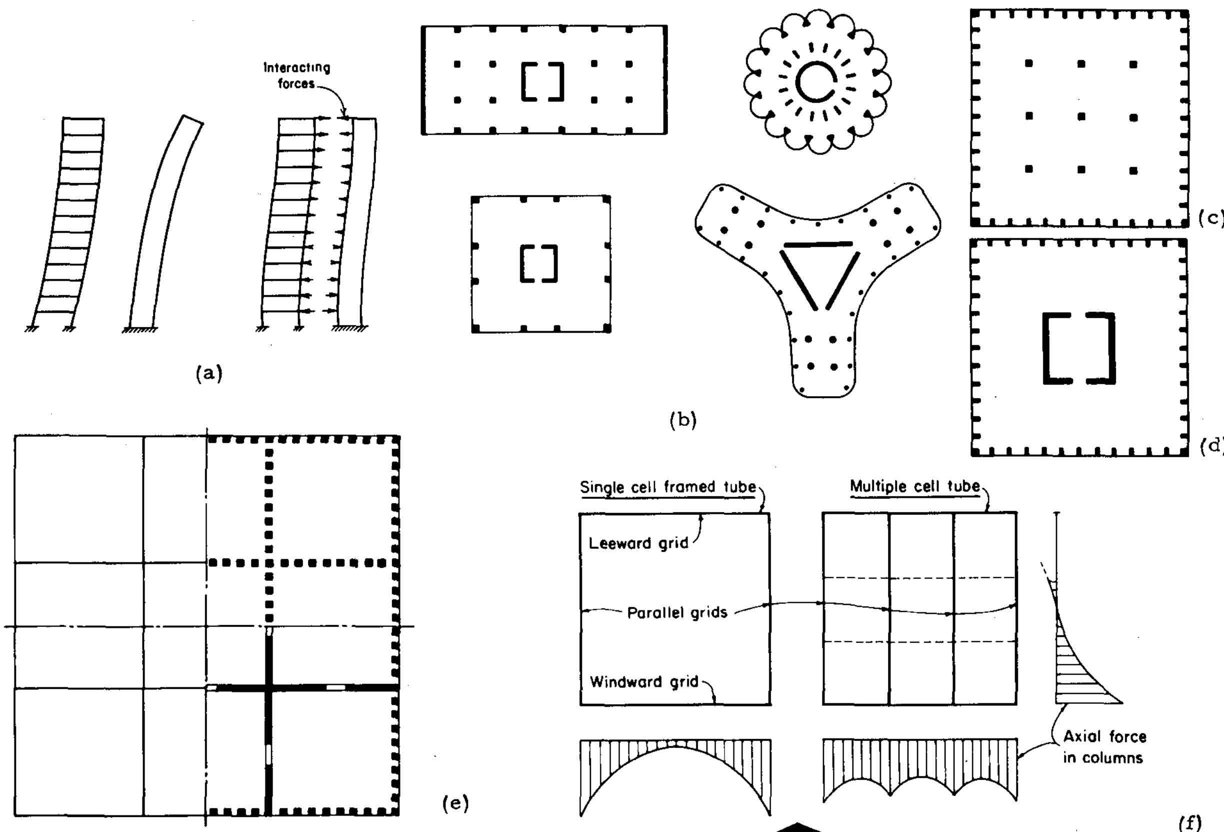


Fig. 1: Lateral Load Resisting Structural Systems

Table 1: Guide to Selection of Structural Systems

Structural Systems	Number of stories*	
	Office Buildings	Apartment Buildings hotels, etc.,
Frame	up to 15	up to 20
Shear Wall (egg crate)		up to 150
Staggered Wall Beam		up to 40
Shear Walls acting with Frames	up to 40	up to 70
Single Framed-Tube	up to 40	up to 60
Tube-in-tube	up to 80	up to 100

*The values given here are based on present day practice as well as trends indicated by current thinking.

For still taller structures, especially where a large plan area is involved, intersecting planes of interior walls or closely spaced column-beam grids traversing the entire width of the building may be used (Fig. 1e). Connecting the interior walls with the exterior peripheral grids reduces the shear lag* across the windward and leeward grids and allows the latter to participate to a greater extent in resisting the lateral load (Fig. 1f). The use of such interior vertical diaphragms, when indicated, essentially produces a vertical multi-cell cantilever box beam [1,3].

Table 1, reproduced from [4], is presented as a guide in the choice of an appropriate structural system for a new building. The ranges of suitability shown may vary somewhat depending upon the use of the building, the story heights, and the design live and wind loads.

EARTHQUAKE RESISTANT DESIGN

Structural Response to Earthquakes

The effects of earthquakes may be due directly to the causative process, such as faulting or volcanic action; or due to the ground motion resulting from the passage of seismic waves. Of the latter effects, two types can be distinguished: one in which dynamic (inertial) effects are predominant; and the other, associated with landslides, soil consolidation or liquefaction triggered by earthquake motions, where differential inertial effects within a structure are negligible. Except in unusual circumstances, most of the damage associated with earthquakes has been the result of dynamic effects, and engineering efforts aimed at designing earthquake-resistant structures are concerned mainly with such effects [5].

The ground motion at a particular site is influenced by three factors: (a) source parameters, such as the earthquake magnitude (energy released), depth of focus and geological conditions at and near the focus; (b) transmission path parameters, i.e., epicentral distance and properties and geological character of the intervening ground; and (c) local site parameters, or the geological configuration and properties of the ground at the site.

The forces induced in a structure by an earthquake result directly from the distortions produced by the ground motion. A simplified picture of the behavior of a building during an earthquake can be obtained by considering Fig. 2 [5]. As the ground on which the building rests is displaced, the base of the building moves with it. However, the inertia of the building mass resists this motion and causes the building to suffer a

*The decrease in the vertical forces transmitted to the columns as one moves from the corner toward the center of a frame subjected to lateral loads.

distortion (greatly exaggerated in the figure). This distortion wave travels along the height of the structure in much the same manner as a stress wave in a bar with a free end. The continued shaking of the base causes the building to undergo a complex series of oscillations.

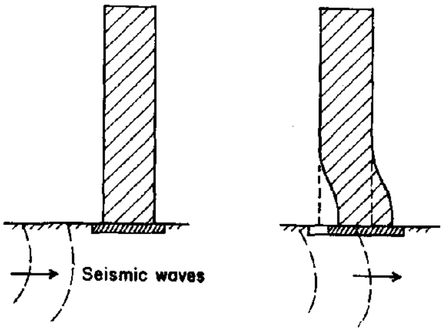


Fig. 2: Effect of Ground Motion on Structure

Design Criteria

The performance criteria implicit in most earthquake code provisions [6] require that a structure to able to:

a. Resist earthquakes of minor intensity without damage (within the elastic range of stresses),

b. Resist moderate earthquakes with minor structural and some nonstructural damage, and

c. Resist major catastrophic earthquakes without collapse.

While no clear quantitative definition of the above earthquake intensity ranges has been given, their use implies the consideration not only of the actual intensity levels, but also of their associated probabilities of occurrence with reference to the expected life of a structure. The quantitative definition of such earthquake intensity ranges would have to consider all the significant ground motion characteristics affecting structural response, i.e., the magnitude of acceleration pulses, frequency characteristics, and duration of the significant portion of the ground motion. The recurrence interval associated with each intensity range would then have to be established for each particular site. The present lack of adequate data on earthquakes renders such an approach beyond immediate realization. The principal concern in earthquake-resistant design is the provision of adequate strength and ductility for the most intense earthquake which may reasonably be expected at a site during the life of a structure, as well as the provision of adequate stiffness for damage control under more moderate earthquakes.

Possible Control of Seismic Input

Certain features (e.g. symmetry, absence of major discontinuities, etc.) in a structure are desirable in that they reduce sharp peak concentrations of earthquake-induced forces. These aspects are discussed in a later section. The generally desirable objective of reducing the seismically induced forces in a structure can logically be pursued further by introducing special devices or mechanisms into the structure. This approach has so far been limited to a very few applications. With the hope that further research will develop the full potential of this possible course, it is briefly reviewed in the following paragraphs.

The schemes which have been proposed in the past to reduce the effect of ground motion on the structure fit mainly into two categories: isolator devices and absorber-damper systems.

Isolator devices--These mechanisms are intended to separate the structure completely from its foundation, using rollers, friction pads, or water. Isolators act essentially as force-limiting devices, since the base shear force in the structure cannot exceed the limiting friction force in the isolating mechanism. To be practical, an isolator mechanism should satisfy the following conditions [7,8]:

1. Relative displacements across the mechanism should be allowed in all directions, but should be limited to certain tolerable values.

2. Post-earthquake residual displacements should be minimized.

3. Wind should not cause relative motion across the mechanism.

4. Preferably, some energy should be absorbed by the mechanism.

5. Impact type forces generated at the end of the operative range of the mechanism should be minimized.

6. Manufacture, maintenance and installation of the mechanism should be as inexpensive as possible.

Requirement 2 above poses the most problems with practical isolation devices. Many apparently feasible solutions to this problem have been proposed [7], and isolators have been used extensively in the case of machine foundations [9]. To date, at least one five-story reinforced concrete building in Mexico City has been isolated from its foundation by a ball-bearing system [10]. A limited amount of research work on isolator systems is currently in progress in the U.S. [11].

Absorber-damper systems--In general, it appears feasible to control the earthquake ground motions input to the base of low, rigid buildings by means of an isolator mechanism, or by taking advantage of the properties of the surrounding soil and its interaction with the foundation [9].

In the case of tall, slender buildings, the control (isolator) system must perform two functions [9,12]. First, it must prevent the build-up of unacceptably large accelerations which may occur as a consequence of resonance in one of the higher modes of the building when it is excited by the high frequency components of the ground motion. Second, it must prevent the development of large deformations in the building which may occur as a consequence of its fundamental mode having been excited by the low frequency components of the ground motion. One way to realize these goals is to increase the damping of the structure to avoid sharp peak values of response. Another way is to confine the energy absorbing function of the structure to built-in special devices or to specially designed portions of the structure, which would absorb and dissipate large volumes of energy through multilinear elastic or elasto-plastic behavior.

The use of elasto-plastic nonlinearity has often been proposed, although such use may not be desirable at all times, since the period of the structure tends to become longer just when the excitation of low frequencies is predominant. Studies have indicated that devices based on the plastic torsion of mild steel bars can provide large energy absorption with adequate fatigue resistance. A practical device based on this principle has been developed and is currently undergoing full-scale tests in New Zealand where it will be incorporated into the piers of a reinforced concrete railway bridge [13]. Another technique for controlling the earthquake energy input to a structure was originally suggested in 1929 [14] and involves the use of a flexible and soft (relatively low yield strength) first story (or lower stories) [15]. During an earthquake, predetermined areas in the lower levels of these structural systems are supposed to undergo bilinear forms of elasto-plastic hysteresis, thereby absorbing most of the earthquake energy. Portions of a building above the soft story then need only be designed for gravity and wind loads, as in nonseismic zones. An 11-story reinforced concrete hospital based on the above principle, designed by ABAM Engineers, has been built in Tacoma, Washington. The presence of a restoring force within the system prevents instability due to large distortions of the soft story. In general, the possible residual displacements associated with elasto-plastic hysteresis may be a drawback of absorber systems based on such hysteresis.

The "double-column" or "multi-column" system, proposed by Japanese researchers, represents one of the ways to produce elastic nonlinearity in a building [7,12]. When multi-columns are used, only the inner columns support the axial and bending stresses until the deformations are large, when the outer columns also share the stresses, thus producing hardening-spring type stiffness. The nonlinear characteristics can be adjusted through the gaps between the inner and outer columns. This system, in conjunction with suitable dampers, is planned for installation in the lower three stories of the 200m tall Yosuda-Kasai Building in Tokyo [12].

The modern trend in research on input control systems is towards 'active control', the aim being to develop control mechanisms which are regulated by electronic signals from sensors of displacements, velocities, accelerations or forces [8,16].

One of the main problems associated with the use of isolator and/or absorber-damper mechanisms concerns their reliability. Use of earthquake simulators (shaking tables) appears to be essential in reliability studies. Tests of full-scale models are not possible with the presently available earthquake simulator facilities, except for very small structures. It is also doubtful whether such tests could be carried out even with the largest conceivable shaking table that could be built in the near future [9].

With or without the use of input control devices, the design of earthquake resistant structures must meet the twin requirements of safety or prevention of collapse and damage control. These requirements will now be discussed separately, and in some detail.

PREVENTION OF COLLAPSE

Need for Ductility

The need for ductility in earthquake resistant reinforced concrete structures has been discussed in depth and with considerable clarity by Paulay and Uzumeri [17]. The following paragraphs closely follow their treatment of the subject.

In most reinforced concrete structures, it is uneconomical to resist the forces, generated during strong seismic ground excitations, within the limits of elastic response of the structure. It is accepted that during rare ground accelerations of large intensity, yielding and consequent plastic deformations may occur at some or all critical areas within the structure. Because prevention of collapse is a fundamental design criterion, it is necessary to ensure that the post-elastic deformations in all parts of the structure can occur while the lateral and vertical load capacities of the structure are substantially maintained.

The ability of a structure to deform past the elastic limit is usually measured in terms of ductility. Ductility in reinforced concrete structures in general is defined as the ratio of a specified distortion at a particular stage of the loading to that at the onset of yielding. With certain restrictions, the term "ductility" may be considered as a useful index of the suitability of a structure for seismic resistance.

Fig. 3 [17] shows the lateral force versus lateral displacement relationship for two structures with identical stiffness, but of differing strengths, responding to the same earthquake. Structure A is able to respond to the given earthquake completely within the elastic range. The maximum deflection corresponding to the full elastic response is Δ_A. In structure B, when the lateral force reaches F_B, the structure reaches its elastic limit at a lateral displacement of Δ_B.

It has been shown [18-20] that whether the response is elastic or inelastic, the displacements which regular medium-to-highrise structures undergo, when subjected to a typical earthquake motion, are of the same order of magnitude. Thus, structure B must be able to deform plastically from B to B', if it is to survive the earthquake. It can be seen that the designer may select a lower strength than the elastic reponse force (F_A), provided that inelastic deformations and the resulting damage are acceptable. The current building codes specify a design load (F_B) 1/3 to 1/6 of the force required to resist the earthquake elastically (F_A).

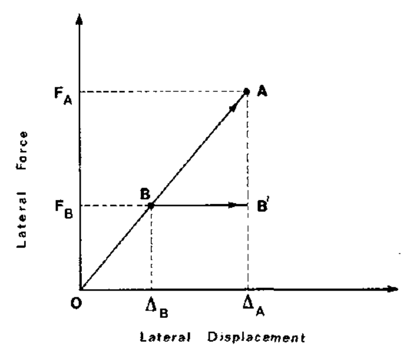


Fig. 3: Force-displacement Relationships of Elastic and Inelastic Structures

This reduction of the strength requirement is justified only if accompanied with design and detailing requirements for the structure to be ductile, so that it can deform plastically without collapse. In the above comparison, Δ_A is the same as Δ_u, the ultimate deflection of structure B, and Δ_B is the same as Δ_y, deflection at yielding of structure B. The ratio $\Delta_A/\Delta_B = \Delta_u/\Delta_y = \mu_\Delta$ is the displacement or system ductility factor for the structure.

Distribution of Ductility Requirements Along Structure

It is important to draw a distinction between the ductility factors associated with the lateral displacement of a structure and local ductility factors. Since the former is achieved through inelastic deformations at the critically stressed portions of a relatively few members, the corresponding local ductility factors are of primary interest in design. Thus, it is worthwhile presenting here some of the more significant results of analytical studies of the earthquake response of frame, wall and frame-wall structures.

Frames--The configuration and relative member stiffnesses of the basic 20-story frame structure considered in a study by Clough and Benuska [21], from which most of the results presented here are drawn, are shown in Fig. 4a. The frames were designed for vertical loads plus the lateral forces prescribed by the Uniform Building Code, 1964 Edition. The yield moments were taken as twice the corresponding computed design values for the girders and six times the corresponding design values for the columns.

In nonlinear dynamic response analyses, the moment-rotation characteristics of the members were assumed to be of the bilinear type, with the post-yield branch having a slope equal to 5% that of the elastic branch. The term ductility factor was defined as the ratio of the maximum rotation at the end of a yielded member, to the yield rotation angle. The yield rotation angle was defined as that corresponding to a moment acting at the end of a simply-supported member having the same section but a span equal to half that of the actual member. The use of a half-span was based on the antisymmetrical mode of deformation of frame members due to lateral displacement.

The results shown were obtained by subjecting the base of the structures to the first 4 sec. of the 1940 El Centro earthquake (N-S component). Other earthquake records with different frequency characteristics may produce results significantly different from those presented.

1. Comparison of linear (elastic) and nonlinear response--Fig. 4b(i) shows that the maximum lateral displacements for both the elastic and the nonlinear frames are approximately equal, as schematically indicated earlier in Fig. 3. This similarity, however, does not imply the development of similar maximum deformations in corresponding members of the two frames. Fig. 4b(ii) shows the girder ductility requirement for the nonlinear case varying from 2 at mid-height to 5 at the top, compared to a maximum-to-yield moment ratio of about 2 for the elastic case. An analysis assuming completely elastic response slightly overestimates the inelastic deformations in the columns (Fig. 4b(iii)). In Fig. 4, as well as in subsequent figures, a ductility factor less than unity indicates the ratio of the maximum moment to the yield moment in a member.

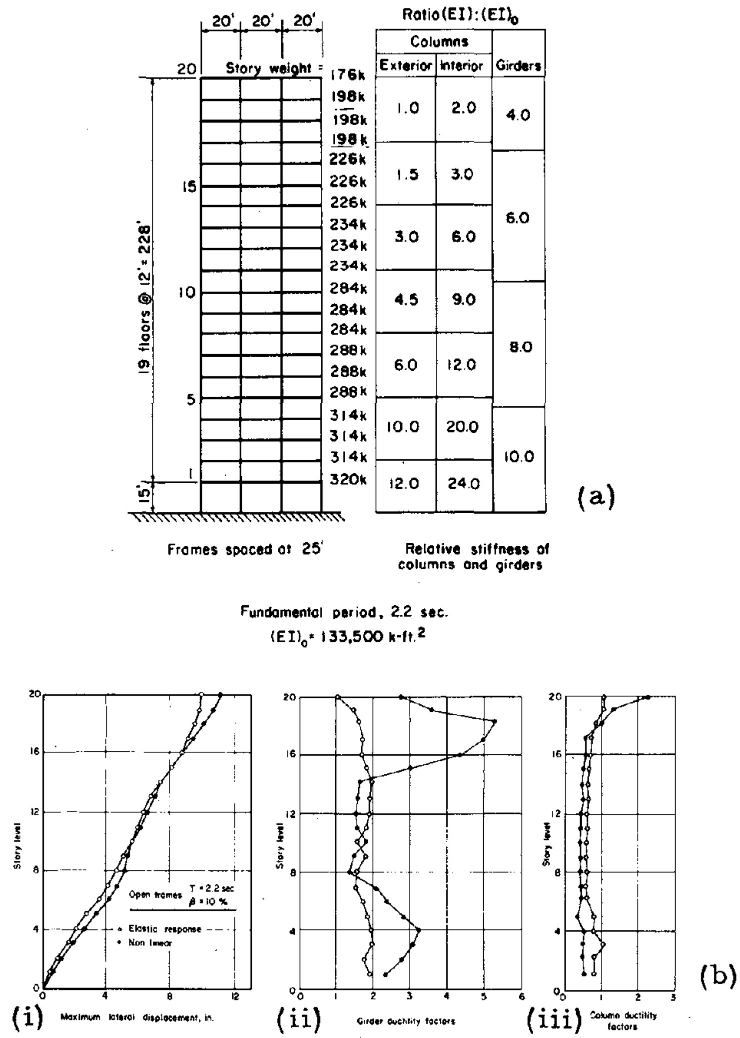


Fig. 4: Seismic Response of Frame Structures

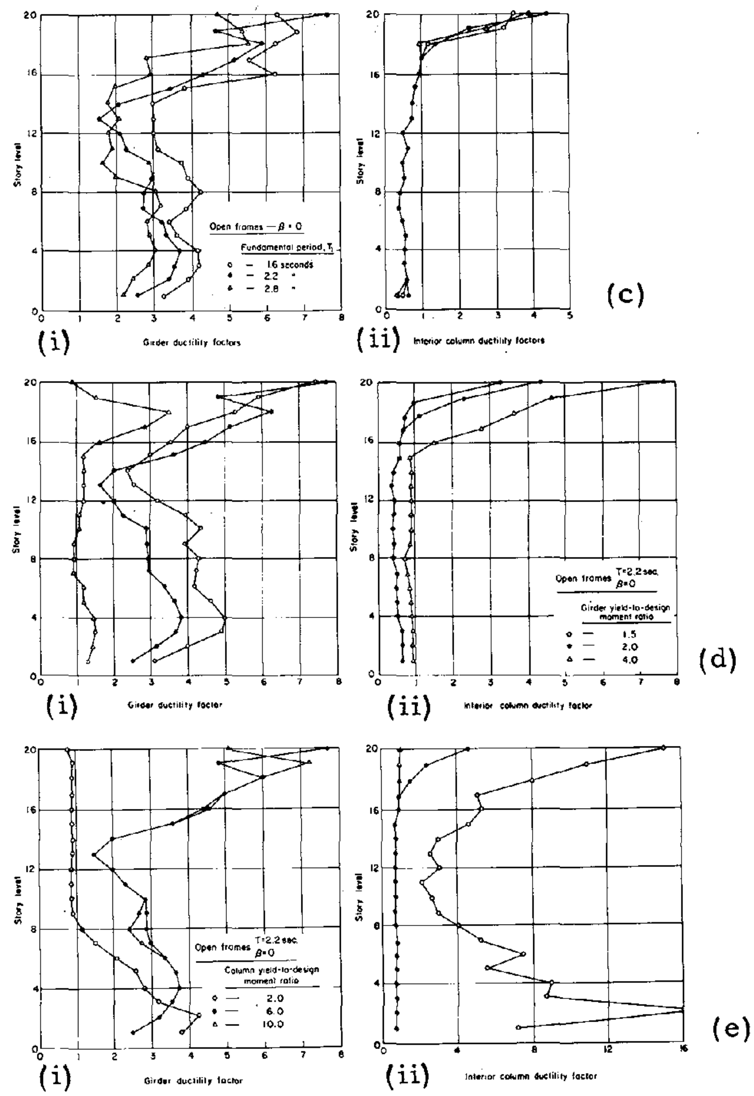


Fig. 4 (cont'd): Seismic Response of Frame Structures

2. Effect of period of vibration--Two 20-story frames having fundamental periods of 1.6 and 2.8 sec. were considered in addition to the standard 2.2 sec. frame. The basic stiffness parameter, $(EI)_o$, was varied to obtain the different periods. The results, shown in Fig. 4c, indicate that there is a slight decrease in girder ductility requirements for the more flexible (long-period) structures. However, a study by Goel and Berg [22] of the response of 10-story, single bay frames to three different earthquake records showed that this particular trend can be reversed in the case of earthquake records characterized by dominant velocity spectrum peaks in the 2-3 sec. range.

A probabilistic study by Ruiz and Penzien [23] of the response of 8-story shear-beam models subjected to a number of artificially generated accelerograms showed that in stiff, short-period structures with a fundamental period of about 0.5 sec., the ductility requirements tend to decrease toward the top of the structure. This contrasts with the variation typical of more flexible frames shown in Fig. 4c where the influence of the higher modes of vibration causes a significant increase in the ductility requirements in the top stories. Ruiz and Penzien also observed that the ductility requirements at the base of a stiff structure are significantly greater than those for a flexible structure subjected to the same excitation.

3. Effect of strength of girders--Three frames were considered: the reference structure with a girder yield-to-design moment ratio of 2.0 and two other frames, identical to the first, except that the yield moments were 1.5 and 4.0 times the design moments.

As expected, the girder ductility requirements decreased with increasing girder strengths. This is shown in Fig. 4d(i). More significant, however, is the fact that the increase in girder strength forced more of the inelastic deformation to occur in the columns, as indicated in Fig. 4d(ii). In general, decreasing the yield strength of one member type, i.e., columns or girders, with respect to another tends to attract inelastic deformation toward the weaker members, resulting in reduced yielding in the stronger member type.

4. Effect of column strength--This variable was studied by considering two frames, having column yield-to-design moment ratios of 2.0 and 10.0, in addition to the reference frame which had a moment ratio of 6.0. Fig. 4e indicates that increasing the column strength beyond that corresponding to a ratio of 6.0 does not materially affect the response. This follows from the fact that the columns in the reference building remain essentially elastic during the response.

Fig. 4e, however, shows that a reduction in column strength can have a significant effect on the distribution of ductility requirements. If the columns do not have a sufficient margin of strength above the design level, most of the inelastic deformations will tend to occur in the columns. Because of the danger of instability associated with excessive yielding in the columns, such a condition should be avoided.

Walls--The results presented are from the report of a recent investigation conducted at the Portland Cement Association (PCA) [24]. The basic structure considered in this study is a 20-story building consisting mainly of a series of parallel walls (Figs. 5a-c).

The stiffness of the wall in the basic building was assumed uniform along the height. A constant wall cross-section throughout the height was also assumed. However, a reduction in the yield strength of sections above the base was included to reflect the effects of axial loads on moment capacity. The building was assumed to be fully fixed at the base. Inelasticity was allowed in dynamic analyses by means of concentrated flexural 'point hinges' which formed at the ends of elements when the yield moment was exceeded at these points. The hysteretic moment-rotation relationship for these hinges was an extended version of Takeda's model [25] which accounts for the observed decrease in reloading stiffness in reinforced concrete members subjected to reversed inelastic loading. A 12-mass model of the 20-story walls was used in analyses (Fig. 5d), with the masses concentrated at each floor level in the first four floors where most inelastic action usually took place.

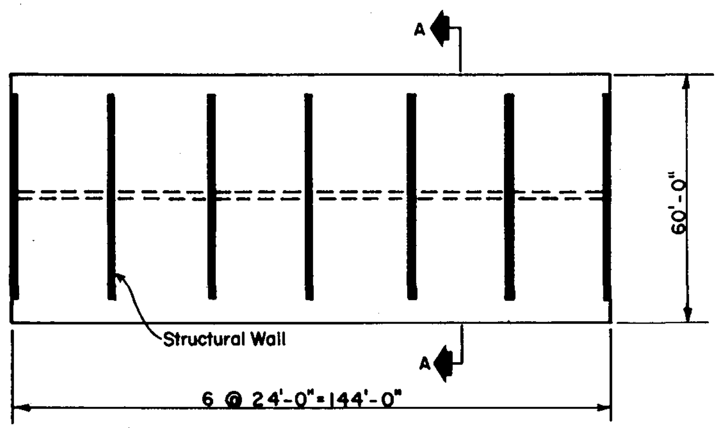
The ground motion used in analyses had the same frequency characteristics as the E-W component of the 1940 El Centro record. The duration of the motion was set at 10 sec. The intensity was normalized to 1.5 times the spectrum intensity corresponding to the first 10 sec. of the N-S component of the 1940 El Centro record.

Ductility was defined on the basis of nodal rotations as being equal to θ_{max} / θ_y where θ_{max} was the maximum computed rotation at the node, and θ_y was the nodal rotation corresponding to yielding at the base.

1. Effect of fundamental period--The effect of the initial fundamental period was investigated using values of 0.80, 1.40, 2.00 and 2.40 sec. to cover the practical range for 20-story buildings. Each period was investigated under varying values of yield strength of the critical section at the base (M_y), in order to examine the relationship between these two major variables and the response quantities. Fig. 5e presents ductility requirements along the height of the walls for M_y = 500,000 in-k. The ductility requirements become greater with decreasing fundamental period (increasing stiffness). Beyond a certain value of the fundamental period, however, the ductility requirements do not decrease significantly with an increase in period.

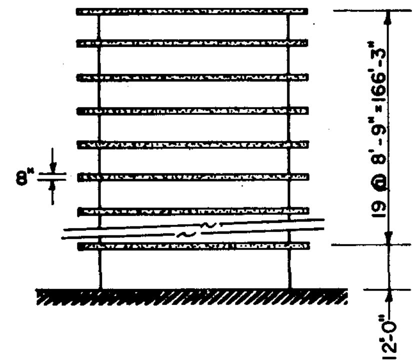
2. Effect of flexural strength--The values considered for the yield strength of the base critical section ranged from 500,000 to 1,500,000 in-k. The results, for the particular case of T_1=1.4 sec. are presented in Fig. 5f. It can be seen that the ductility requirements increase significantly as the yield level decreases.

Frame-Wall Systems--The results presented here are from the Clough-Benuska study referred to earlier [21]. Fig. 6a shows the relative stiffnesses of the members of the standard structure in terms of a reference $(EI)_0$. The value of $(EI)_0$ has been adjusted to give the standard



PLAN

(a)

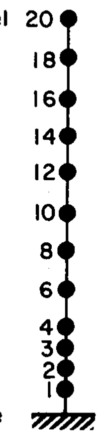


SECTION A-A

(b)

Number of stories		20
Height		178.25 ft.
Weight (for mass computation)		4370 k/wall
Stiffness EI for T_1 =	0.80 sec	6.34×10^{11} k-in^2/wall
	1.40	2.052×10^{11}
	2.00	0.990×10^{11}
	2.40	0.684×10^{11}

(c)



(d)

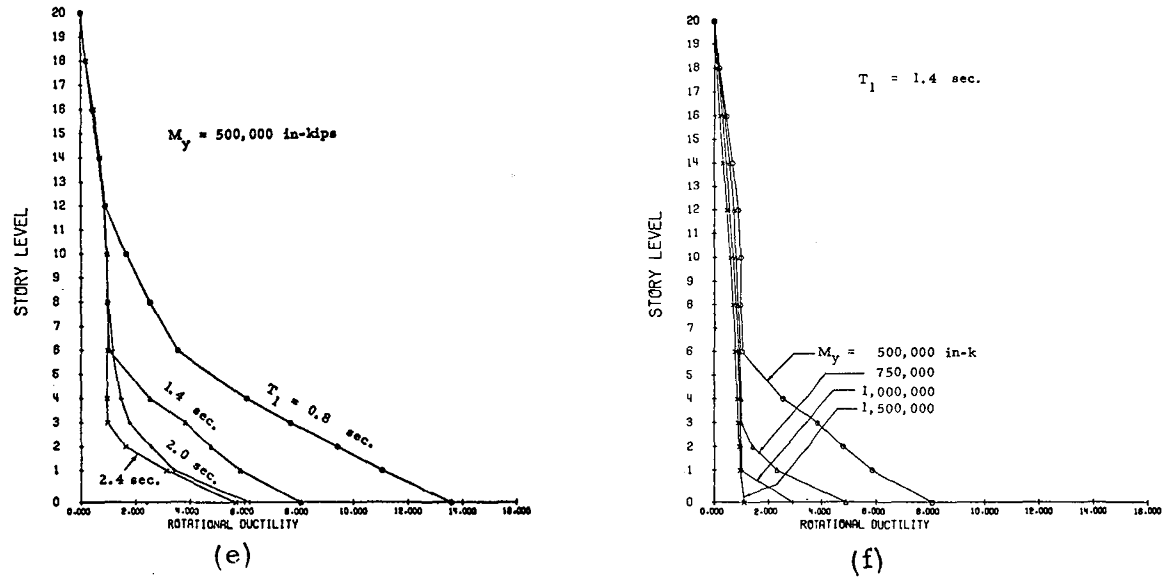


Fig. 5 (cont'd): Seismic Response of Isolated Structural Walls structure a fundamental period of 2.2 sec. As with the frame building discussed earlier, the design moments were determined by a computer analysis for the static vertical and the code seismic forces.

1. Effect of design assumptions concerning distribution of lateral loads between frame and structural wall--Designs corresponding to three different ways of distributing lateral loads, found in practice, were considered. A first design was based on the assumption that the entire lateral load was carried by the structural wall. The frame for this building was designed only for vertical loads, with the girder moments being uniform and the column moments increasing from top to bottom. A second design was based on the Uniform Building Code provisions requiring the frame to be designed for (at least) 25% of the total lateral forces. This led to girder and column moments, both of which increasd from top to bottom. A third design was based on the true interactive behavior of the frame-wall system. In this case, the girder and column moments were largest at mid-height and decreased both upward and downward. In each case, the ratio of yield-to-design moments was set equal to 2 for the girders and 6 for the columns and walls. In all cases, the reference stiffnesses were adjusted to yield a fundamental period of 2.2 sec.

The girder and column ductility requirements corresponding to the three buildings considered are shown in Fig. 6b. Also shown is a curve corresponding to the 25% lateral-load frame building with the structural wall hinged at the base. The very favorable distribution of strength in the interaction frame building, resulting in significantly lower ductility requirements for girders over the entire height and for columns at the top, is evident. The relatively low design strength of the frame in the gravity-load frame building is reflected in the high girder ductility requirements. It is worth noting that designing for frame-wall interaction tends to eliminate yielding of the columns at the top stories.

Fig. 6b shows that the ductility requirements (M_{max}/M_y for ductility ratios less than unity) in the structural walls for the four buildings considered are roughly of the same order of magnitude. For the yield-to-design moment ratio assumed, none of the structural walls was stressed beyond the elastic range.

2. Effect of period of vibration and frame-to-wall stiffness ratio--Two structures with fundamental periods of 1.6 and 2.6 sec. were considered in addition to the reference 25% lateral-load frame building, having a fundamental period of 2.2 sec. The 2.6 sec. building had a 10-ft wide structural wall with a stiffness ratio relative to the wall in the reference building of 0.2, while the 1.6 sec. building had a 38-ft wide wall and a stiffness ratio of 5.0.

Fig. 6c shows the girder and column ductility requirements. There is no significant difference in column ductility requirements among the three buildings. A slight decrease in girder ductility requirements occurs in the stiffer (shorter-period) structures. This trend is contrary to that observed in open frame structures. It should be realized that the periods of the three structures considered differ, not because of a change in stiffness of the frames (as was the case with the frame structures

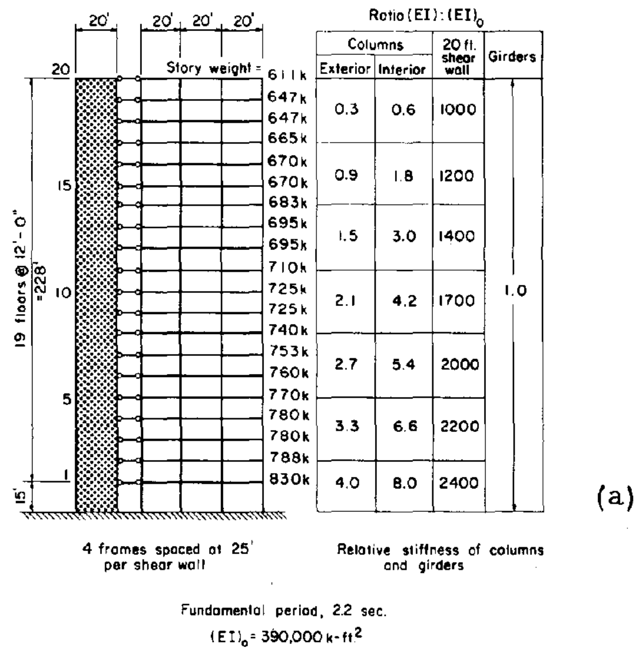


Fig. 6: Seismic Responses of Frame-Wall Structures

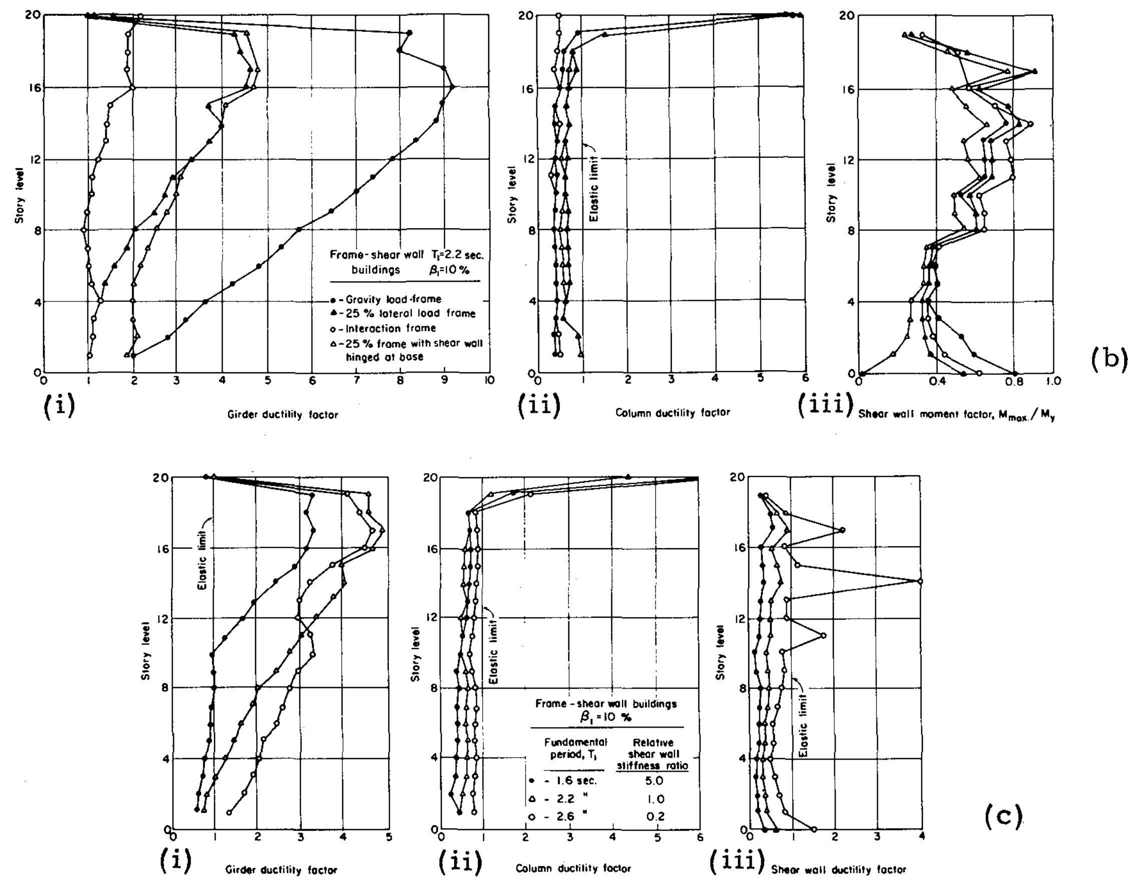


Fig. 6 (cont'd): Seismic Responses of Frame-Wall Structures discussed earlier), but because of a change in the width and hence the stiffness of the structural walls. In all three structures, the frame portions were identical. The observed difference in girder ductility requirements can thus be interpreted as reflecting the effect of the wall-to-frame stiffness ratio rather than of the period of vibration.

A plot of the structural wall ductilities, shown in Fig. 6c, indicates a decreasing ductility requirement for the stiffer structures. More important, however, is the relatively large ductility requirement indicated for the 2.6 sec. structure, compared to the elastic behavior of the other two structures. This points to the potential danger of rupture in such stiffening elements in structures with low wall-to-frame stiffness ratios.

Design for Prevention of Collapse

The design of a structure for prevention of collapse usually consists in proportioning and detailing the critical regions such that they possess adequate strength and ductility. The discussion in this section is thus focused on the critical regions, rather than on the structure as a whole.

One important design consideration affecting the integrity of the entire structural system is the provision of 'multiple lines of defense' This can be accomplished with a high degree of static indeterminacy and with the establishment of an advantageous sequence in the propagation of yielding. This aspect is discussed in a later section.

While there have been attempts to relate code-specified minimum system ductilities and local ductility requirements in the case of frames [26] and cantilever walls [17,27], the best way to assess the local ductility requirements in the critical regions of a particular class of structures, corresponding to a specified earthquake intensity, is to carry out dynamic inelastic analyses of structures representative of the class under various combinations of structural and ground motion parameters. This was done at PCA in the case of isolated wall structures [24,28], resulting in charts such as the one illustrated in Fig. 7. The chart gives the required ductilities, based on nodal rotations at the first floor level, in 20-story structural walls under an earthquake with a spectrum intensity equal to 1.5 times that of the first 10 sec. of the N-S component of the 1940 El Centro record. The ductility requirements decrease with increasing flexural yield strengths of the base critical section, as well as with increasing periods. There will, in practice, be an upper limit on the period or flexibility (as discussed later). One is thus faced with a situation of trade-off between flexural strength and ductility requirements.

It must be understood that the critical regions have to be designed such that the required ductilities are attainable in the presence of the shear and the axial stresses which the regions are called upon to carry. The presence of shear has a decidedly adverse effect on ductility. While axial loading is known to have a detrimental effect on the curvature ductility of a section [29], its effect on the rotational ductility of a member segment is not necessarily harmful. This is because the presence of axial loads results in an enlarged concrete compression zone capable of transmitting shear. This has a delaying, if not preventive, effect on the

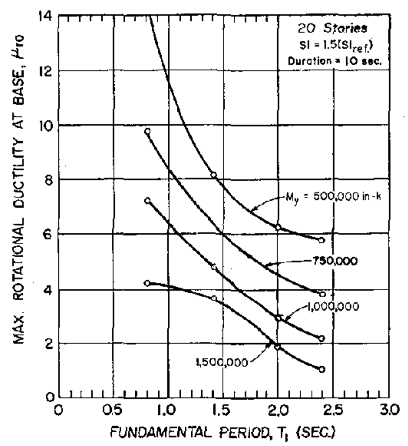


Fig. 7: Flexural Strength and Ductility Demands in Isolated Walls

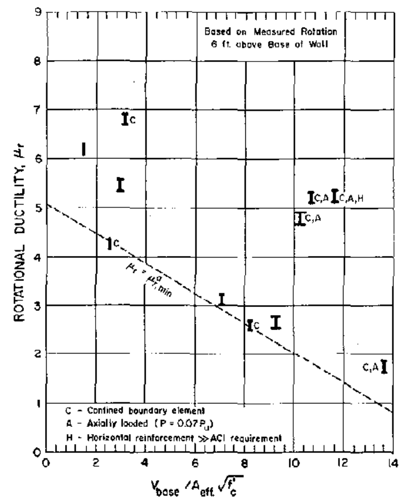
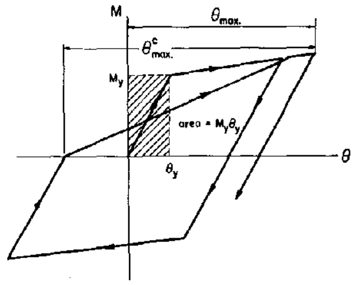


Fig. 8: Rotational Ductilities Available in Hinging Regions of Isolated Walls

possible occurrence of nonductile shear failure. Rotational ductility is thus enhanced by axial loading, particularly in the presence of high shear. Axial loading, to have any beneficial effects, however, has to be limited in magnitude to rather moderate levels. High axial loads, tending to cause compression failures, are bound to have harmful effects. Fig. 8, based on recent tests carried out at PCA [30], may serve as a guide to rotational ductilities available in reinforced concrete wall segments. The beneficial effect of confinement reinforcement on ductility should be noted. More experimental research is needed in this area to establish minimum available ductilities as functions of axial and shear stresses, under various combinations of sectional parameters (shape, amounts of flexural, confinement and shear reinforcement, etc.). Such research must also extend to critical regions of frames and frame-wall systems.

In [28], the possibility was raised that designs based on a comparison of just one measure (e.g., rotational ductility) of deformability demand and deformation capacity may not be entirely safe, particularly since the estimates of deformation capacity are usually based on laboratory loading histories which are different from those experienced by critical member segments under seismic conditions. Thus, in addition to rotational ductility as defined in Fig. 9, three other measures of deformation as well as energy dissipation (also defined in Fig. 9) were considered. Based on a comparison between estimated available and required values of the various quantities under severe earthquake conditions, it was determined [28] that designs satisfying minimum deformability (energy dissipation) requirements in terms of rotational ductiity will also be safe with regard to the other measures of deformation and energy dissipation.

In addition to ductility, sufficient shear strength for the critical segments must also be provided for in design. This may not always be simple, since repeated reversed loading of reinforced concrete member segments in the inelastic range may lead to a reduction in their shear resistance. There is a paucity of test results on which one can base reasonable estimates of the shear capacity of reinforced concrete member segments subjected to repeated reversing loads of large amplitude [31]. Partly because of this, and partly out of concern for ductility as well as energy dissipation capacity (which suffers because of stiffness degradation caused by moment reversals in the presence of moderate-to-high shear), Paulay [32] has made the following suggestion for the case of tall (height/depth >3) structural walls: "Where it is essential that the lateral and gravity strength be maintained in a ductile manner, ...every attempt must be made to suppress a shear failure. This is only possible if the shear force, associated with the maximum possible flexure strength of the critical section, taking into account the increased yield strength of the flexural reinforcement due to strain hardening, is provided for in such a way that the shear (web) reinforcement will not yield." The following suggestions concerning low (height/depth <2) structural walls have also been made:



Measures of Deformation:

rotational ductility, $\mu_r = \dfrac{\theta_{max.}}{\theta_y}$

cyclic rotational ductility, $\mu_{rc} = \dfrac{\theta^c_{max.}}{\theta_y}$

cumulative rotational ductility, $\Sigma\mu_{rc} = \dfrac{\Sigma\theta^c_{max}}{\theta_y}$

cumulative rotational energy, $\Sigma A_r = \dfrac{\text{cumulative area under } M-\theta \text{ loops}}{M_y\theta_y}$

Fig. 9: Definitions of Ductility and Energy Dissipation Capacity

(a) If a ductile (i.e., flexural) failure mechanism is desired, then the nominal shear stresses, associated with the maximum possible flexure strength of the critical section, must be moderate say, $v_u < 5\sqrt{f'_c}$ psi.

(b) Because the flexural failure mechanism is associated with large cracks, no reliance can be placed on the concrete within the hinging region in contributing towards shear strength. Consequently, in the hinging region the whole of the shear force should be resisted by stirrups.

Bertero and Popov [33,34] recommend that flexural members in general be designed such that their maximum bending strength does not require the development of maximum average nominal shear stresses beyond $3.5\sqrt{f'_c}$ psi. If it is not possible to keep the nominal shear stress below this level, special web reinforcement beyond that required by present code provisions should be used. Even then, the maximum nominal shear stress should not exceed $6\sqrt{f'_c}$ psi if two or more load reversals at a displacement ductility ratio of 4 or greater (for the member) is expected. The maximum nominal shear stress, in any case, should preferably be confined to a value considerably lower than $10\sqrt{f'_c}$ psi.

DAMAGE CONTROL

The provision of adequate local ductilities and shear strength in all critical regions will not only minimize the probability of collapse, but will usually also minimize earthquake damage to the structural elements of a building. Careful attention must be paid to the detailing of joints and to proper anchorage of all reinforcement. In addition, considerations such as (1) the avoidance of unnecessary torsion and force concentrations, (2) proper tying together of structural elements, (3) prevention of hammering, and (4) taking proper account of stiff infills in spaces between frame elements or columns, are also important. These considerations are discussed in the next section. The discussion here is on damage to nonstructural elements, which is of utmost concern, since such elements represent a major portion of the cost of residential and office buildings.

The magnitude of interstory horizontal deformations appears to be the prime factor determining the amount of earthquake damage to nonstructural elements.

Force-Deformation Characteristics of Nonstructural Elements

Nonstructural elements can be made of brittle or ductile materials, each characterized by its own response to loading.

Brittle elements such as unreinforced masonry partitions, glass panes, etc. fail abruptly after reaching their maximum strength. Depending upon the magnitude of their deformation before sudden brittle failure, they can be either relatively rigid (asbestos cement sheets) or relatively flexible (gypsum drywall panels).

Ductile elements reach their maximum strength and continue to deform while maintaining an acceptable load level. Many of the brittle materials can be made into ductile elements, either by reinforcing them (i.e., reinforced masonry), or by proper assembly of units (i.e., walls made of individual gypsum drywall panels with flexible connections between them).

While for some nonstructural materials and assemblies the force deformation characteristics are known, no information exists for many other elements. This lack of information makes it difficult, if not impossible, to establish rational limitations on interstory distortions. Research is needed to establish force-deformation characteristics for all nonstructural elements incorporated into earthquake-resistant buildings.

Design and Detailing for Damage Control

If the nonstructural elements are ductile and are thus able to distort and accommodate the elastic and plastic distortions of the structure without cracking or breakage, then no special detailing is required; these elements will not suffer any significant earthquake damage.

Where brittle nonstructural elements are used, they can be protected against earthquake damage by using them in conjunction with rigid structures having interstory deformations restricted to a level which can be tolerated by the brittle elements. This can be accomplished in buildings incorporating structural walls, except for the hinging region in which shearing type deformations may be large. Within the anticipated hinging region, special detailing for brittle nonstructural elements may be required.

In flexible structures (frames) with expected interstory deformations larger than the damage deformation capability of brittle nonstructural elements, such brittle elements should be detailed so as not to be strained when the frame distorts in an earthquake. Partitions can be made "floating", window panes can be embedded in neoprene gaskets, and mechanical appurtenances can be specially detailed. The amount of expected deformation can be determined from analysis by considering the combined elastic and inelastic story deformations.

Although generally the only adverse effect of the required special details is to add to the cost, there are instances where performance is affected, as when accoustic problems are caused by floating partitions.

PLANNING AND DESIGN CONSIDERATIONS

A reasonably good basis for a preliminary design of an earthquake-resistant building would be a structure proportioned to satisfy the requirements of gravity and wind loading. The planning and layout of the structure, however, must be undertaken with proper consideration of the dynamic character of earthquake response. Thus, modifications in both configuration and proportions to anticipate earthquake requirements may be incorporated immediately into the design for gravity and wind. The following are some of the design considerations.

1. Drift limitation--A limitation on drift or lateral deflection due to wind is the principal criterion used in assessing the proper lateral stiffness to be built into tall buildings, and may determine the type of structural system to be employed. The use of a maximum allowable drift is based on the need to limit to safe or tolerable levels the effects of lateral sway on (a) the stability of individual columns as well as the structure as a whole, (b) the integrity of nonstructural elements, and (c) the comfort of the occupants. The precise relationship between drift due to wind and the above three factors remains to be established. To date only the Uniform Building Code [35], and the National Building Code of Canada [36], among the North American model building codes, specify a maximum drift of H/500 (H = building height), corresponding to the design wind loading. Also, ACI Committee 435 recommends a drift limit of H/500 [37]. The present day design of tall reinforced concrete buildings containing structural walls provides extremely rigid structures [38] with a drift (computed by advanced methods) between H/1000 and H/2500, depending on the slenderness ratio of the building and the number and layout of walls.

Basically the same considerations as mentioned for wind enter in aseismic design, although one might expect slightly more liberal drift limitations under major earthquakes. For severe earthquake motion, the principal consideration insofar as drift is concerned is the stability of the structure under the action of gravity loads when undergoing large lateral displacements. The SEAOC Code [6] mentions an allowable drift due to the specified earthquake forces twice that allowed for wind. In applying such a limit, a distinction should be made between the drift produced by the code-specified static forces and the dynamic lateral displacements corresponding to a particular earthquake. The latter could be several times larger than the former [21]. It also follows from the previous section that the need for damage control may require a limit on the interstory drift as well, although no specific limits have been suggested.

Fig. 10 based on the PCA study on isolated walls [28] shows the maximum drift and interstory drift in 20-story walls subjected to intense earthquakes (spectrum intensity = 1.5 times that of the first 10 sec. of the N-S component of the 1940 El Centro record), as functions of the fundamental period and the rotational ductility available in the critical region at the base (extending to the first floor level). The displacements shown are envelope values of the displacements caused by a number of earthquakes of varying frequency characteristics. It can be seen that if and when suitable limits on the drift as well as the interstory drift can be decided on, a corresponding allowable upper limit on the fundamental period can be established. The latter can, in turn, be translated into an allowable lower limit on the flexural stiffness.

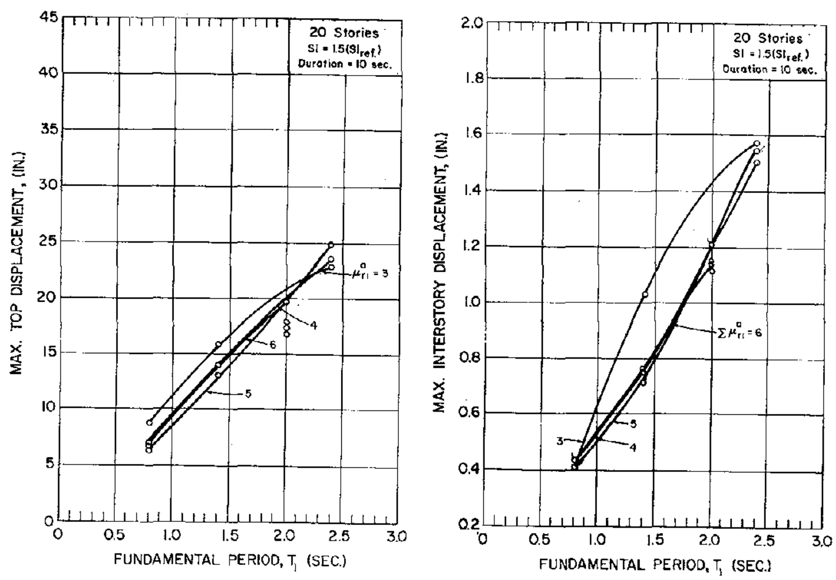


Fig. 10: Maximum Lateral Displacements and Interstory Displacements in Isolated Structural Walls

2. Avoidance of unnecessary torsion and force concentrations--A building which is simple in both plan and elevation, with a minimum of setbacks or changes in section, is generally preferable to an irregularly shaped structure. This is because the effects of force concentrations which occur at major discontinuities in either geometry or stiffness even under static loading, tend to be aggravated under dynamic conditions. The required ductility at such regions of discontinuity is usually substantially greater than at other portions of a structure.

Although it may not be practical to plan a fully symmetrical building, any effort to reduce the eccentricity of the effective inertial force due to the noncoincidence of the centers of mass and of rigidity will pay off in reduced torsional stresses, which can be critical in corner columns and end walls. Locating the major stiffening elements near or along the plan periphery of a building substantially improves the torsional resistance of the structure.

3. Building multiple lines of defense--This aspect has been lucidly discussed by Paulay [32] , on whose treatment the following two paragraphs are based.

Cantilever walls can provide excellent resistance against lateral loads and can greatly reduce deflections. However, for seismic conditions they offer only a single line of defense. Should a large excitation require yielding, this is likely to cause permanent deformations near the base, and may lead to early misalignment in the building. Regular arrangements of openings in cantilever walls enable coupled walls to be formed. In seismic areas it is essential that the coupling beams rather than the walls form the weaker elements. With suitable detailing, coupled structural walls can be both efficient in load resistance and sufficiently ductile. Energy dissipation, when required, can be well dispersed over the height of the structure, and thus several lines of defense may be mobilized when extreme displacements are imposed on a building.

In general, a high degree of static indeterminacy is desirable in earthquake-resistant buildings. It is further desirable that an advantageous sequence in the propagation of yielding be established, so that damage in repairable and less critical areas will occur first. The principal gravity load carrying units will then receive the greatest degree of protection. The designer must establish an intelligent hierarchy in the most probable strength levels which he intends to provide for each structural component.

In connection with the above, it is interesting to note the current approach to seismic design in New Zealand, as embodied in New Zealand Standard 4203:1976. This code requires that [39] buildings expected to undergo flexural ductile yielding be designed by a procedure called capacity design. In the capacity design of earthquake-resistant structures, energy-dissipating elements or mechanisms are chosen and suitably designed and detailed, and all other elements are then provided with sufficient reserve strength capacity to ensure that the chosen energy-dissipating mechanisms are maintained throughout the deformations that may occur.

4. Tying together of elements--The need to adequately tie together all the structural elements making up a building, or a portion of it which is intended to act as a unit, cannot be overemphasized. This applies to superstructure as well as foundation elements, particularly in buildings founded on relatively soft soil. Here, attention should be focused on the design of the segments of elements at and near the joints, since these are generally the regions which are most critically stressed.

Adequate connections should be provided across construction joints if they are required between parts of a building or between the main portion of a structure and an appendage, e.g., stairway enclosure, carport, etc..

5. Prevention of hammering--The different portions of a building should either be tied together adequately or separated from each other by a sufficient distance to prevent their hammering against each other.

Expansion or similar joints used to separate parts of a building which differ considerably in height, plan size, shape, or orientation should be sufficient to allow the components to sway independently of each other without impact. Any required passageway, corridor or bridge linking structurally separated parts of buildings should be so detailed as to allow free, unhindered movement during an earthquake.

In order to avoid hammering between adjoining buildings or separate portions of a building when vibrating out of phase of each other, a gap (perhaps filled with readily crushable material) equal to from four to six times the sum of the calculated lateral deflections of the two structures under the design (code) seismic forces, or the sum of the maximum deflections of the two structures as indicated by a dynamic analysis, would be desirable.

6. Infilled frames--The use of very stiff walls to fill the spaces in relatively flexible frames should be considered carefully during the preliminary design stage. The presence of rigid infill (having corresponding strength) causes the infilled frames to behave like cantilevers, thus totally changing the behavior of the frame elements.

If the infilling material is intended to act in combination with the enclosing frame, then it should be designed and constructed to ensure this composite action. Proper reinforcement and connection to the enclosing frame are essential. The analysis should likewise consider the increased stiffness and modified behavior of the infilled frames.

If the infill is made of fairly brittle material, such as glass or hollow brick masonry, and is not expected to contribute significantly to the lateral resistance of the frame, then it should be effectively isolated from the surrounding frame by gaps or readily crushable or yielding material to allow sufficient relative movement between the frame and such elements. The disastrous effect of deformation incompatibility between flexible frames and brittle infills has been observed in many earthquakes.

7. Reduction in the clear height of columns--The effect of introducing low walls between columns, as shown in Fig. 11, should be

noted. The reduction in height of the columns increases their stiffness with respect to bending in the plane of the wall. This will cause the columns to be subjected to greater horizontal shears than they would be expected to develop if the walls were absent. This is in addition to the effect which a decrease in the period of vibration of the structure--due to the increase in stiffness of the columns--will bring. The reduction in height also reduces the lateral deformation capacity of the columns in the plane of the wall. The use of such walls without allowing for their effects on the columns has been known to cause severe distress in portions of the columns above the wall.

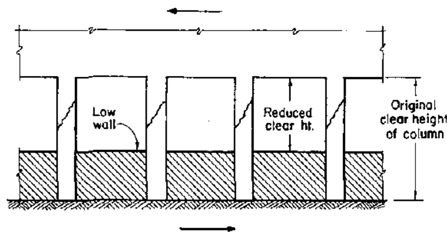


Fig. 11: Effect of Introducing Low Walls Between Columns

SEISMIC PERFORMANCE OF REINFORCED CONCRETE STRUCTURAL SYSTEMS

The performance of reinforced concrete buildings in many past earthquakes [40] has demonstrated that such buildings, when properly engineered, can withstand severe earthquakes not only without collapse, but also without serious damage to either structural or nonstructural elements. Collapse of reinforced concrete structures, as well as failure of individual structural elements or their connections, when such occurred, could be traced either to a lack of adequate strength and ductility, inadequate construction procedures, or to a lack of attention to the design considerations enumerated in the previous section. Damage to nonstructural elements, on the other hand, could be traced either to insufficient structural stiffness or to a lack of proper detailing of the connections between structural and nonstructural components.

While the foregoing, in general terms, affirms the adequacy of the current aseismic design procedures, serious deficiencies exist in the current capability for predicting the actual seismic performance of a structure. This has been clearly pointed out by Bertero [41] whose treatment of the subject is closely followed in this section. In the case of earthquake excitations, it is usually necessary to predict the force-displacement relationships for each story of a structure. The lateral displacement at any story (ΔH_i) can be expressed as a function of the gravity forces acting on the structure [G(t)], and the dynamic characteristics of the soil and the structure, which can be represented symbolically by the period [T(t)] and the damping coefficient [ξ(t)]. Thus [41] :

$$\Delta H_i(t) = f\,[G(t),\ T(t),\ \xi(t)] \qquad (1)$$

An analysis of the parameters involved in Eq. (1) indicates the following difficulties with the prediction of seismic response [41]:

1. All the parameters are time-dependent, except that the gravity forces usually remain practically constant for the duration of an earthquake. Thus, (a) the effect of the inertia forces developed at the masses cannot be neglected, (b) the rate of loading may be high enough to

affect the static-mechanical characteristics of the materials, on the basis of which the dynamic characteristics of the structure [$T(t)$, $\xi(t)$] are usually predicted, and (c) the possibility of low-cycle fatigue as well as incremental collapse must be considered.

2. The inertia forces depend not only on $U_g(t)$ but also on $\Delta H_i(t)$, $T(t)$, $\xi(t)$. This interaction between structural response and the forces themselves poses particularly intricate problems.

3. Lateral displacements, $\Delta H_i(t)$, depend on the ground motion occurring at the foundation of the building, rather than on the free-field ground motion, $U_g(t)$. The actual ground motion depends on soil-structure interaction. This interaction affects not only $\Delta H_i(t)$, but also $T(t)$ and $\xi(t)$.

4. *Structural elements interact with one another and with nonstructural* elements in a complex manner which depends on the detailing of their joints and connections.

5. Since inelastic deformations are not single-valued functions of stress, but are dependent upon the prior deformation history, a knowledge of both the critical loading combination and the history of loads is necessary.

The above difficulties are a clear indication of the need for experimental studies of actual buildings under real earthquakes. In fact, several reinforced concrete buildings and their surroundings around the world have been instrumented, and are currently under observation. Unfortunately, however, one cannot afford to wait for extreme earthquake excitations to occur in the vicinity of these few buildings to learn about their inelastic behavior. Ideally, the next best approach would be the testing of actual instrumented structures under simulated extreme excitations. However, the technical and economic problems associated with the generation of such extreme excitations have so far proved intractable. A more feasible approach is to reproduce ground motions by means of controllable shaking tables. Many small and a few medium-size simulator facilities are already in use. Although the research potential of these facilities is excellent, they can only test small-to-medium-scale models of complex structural systems. These models are usually inadequate to investigate in detail the actual dynamic characteristics and failure mechanism of the prototype. Furthermore, the reproduction of the actual ground motion (three components of displacement) is not easy. Most shaking tables reproduce only one of the three components at a time.

The foregoing indicates that at present the testing of actual complex structures under extreme dynamic excitations, real or simulated, is not altogether feasible. The most logical alternative is to subject actual structures or large-scale models to equivalent pseudo-static forces intended to induce effects similar to those of real dynamic excitations. Since in reality the inertia force at each concentrated mass varies with time, depending on the interaction between the real dynamic excitation and the dynamic characteristics of the building, the simulation of the actual inertia forces by simple static forces is a very difficult problem. One possible solution is to simulate what can be considered the critical combination of inertia forces that can be developed at a certain time. Rational selection of this critical combination requires integrated analytical and exerimental studies (e.g., the PCA investigation reported in [24,28,30]), because it will vary depending on what one is interested in studying. Further, even if a rational combination of inertia forces can be selected, the problem of how to vary the magnitude of these forces still remains, since the behavior of reinforced concrete is very sensitive to the loading path. This problem has usually been solved by adopting arbitrarily selected load sequences. In spite of these shortcomings, however, tests of structures and structural models under pseudo-static loading have yielded much valuable information.

Another approach which has proved useful, attempts to predict the response of a complete structural system from results obtained in studies of its structural elements. Concerning this approach, Newmark and Hall [42] have commented: "The strength of the combined system, the damping in it and the mode of failure can in some cases be inferred from the properties of the individual element; however, these members interact on one another in a complex way and in different ways for different types and directions of loading, and the interaction is a problem which must be taken into account in detail much more accurately than has been the case in the past if adequate lateral resistance to dynamic forces is to be achieved."

Bertero [41] has presented a thorough review of experimental studies that have been carried out on the behavior of reinforced, prestressed and partially prestressed concrete structures and their elements. Such studies will be analyzed under topics VII, VIII and X of this workshop. The prediction of earthquake performance of concrete structural systems need not, therefore, be discussed any further in this report.

PRESTRESSED (INCLUDING PRECAST) CONCRETE SYSTEMS

Prestressed concrete is seldom used in primary resistant structures against repeated loading conditions as severe as those expected to be caused by major earthquakes [9]. The principal reason for this has been the shortage of experimental evidence on the behavior of prestressed concrete members and member assemblies under such loading conditions. Prestressed elements, when utilized, have been used in conjunction with conventional frame, wall and frame-wall systems. Very often, only certain elements in the systems have been prestressed. Both cast-in-place and precast prestressed elements have been used. Precast wall and slab elements (mostly prestressed, occasionally conventionally reinforced) have been used in a structural system equivalent to, yet different in some respects from, the conventional cast-in-place wall construction--the so-called Large Panel structural system. A particular variation of this type of construction, the box-type structure, is fairly extensively used for low-rise buildings. Neither the available ductility nor the energy dissipating mechanism in this structural type has been fully explored as yet.

Prestressed concrete structures in general and precast structures in particular will be discussed under topics IX.1 and IX.2, respectively, of this workshop. Thus, the treatment here is limited to a few general remarks pointing out a number of features peculiar to prestressed (including precast) concrete.

Blakeley [43] has produced a comprehensive historical review of the seismic resistance of prestressed concrete structures and structural elements. According to him, most structures containing prestressed elements which have been subjected to earthquakes have performed well. Failures which have occurred appear to have been due mainly to failure of the supporting structure or of the connections. However, there is relatively little information on the behavior of fully framed prestressed concrete structures under strong earthquakes. The recent Romanian earthquake was the first in an area with a large number of precast structures (up to 9 stories high), and they performed well [44]. However, their response to this earthquake was mostly within the elastic range, making it difficult to reach an assessment as to their seismic resistance. For precast structures, the methods of joinery and their reserve strength and ductility present difficulties not always encountered in poured-in- place concrete [45]. These problems assume prime importance in the design of such structures.

Based on the experimental and analytical studies reviewed by him, Blakeley [43] has pointed out that:

1. Although the energy absorbed by a prestressed concrete member could be the same as or even larger than that absorbed by a similar reinforced concrete member, the greater elastic recovery of the prestressed member will result in a lower energy dissipation for cyclic loading. This is a drawback in seismic design. However, little is known of the energy-dissipation capacity of prestressed members under high-intensity cyclic loading. The energy dissipation would be greater for partially prestressed members once the mild steel yields, but the joints of such members present particular difficulties for precast construction.

2. Because of the lower energy dissipation capacity, and also because of the lower damping applicable to prestressed concrete relative to reinforced concrete, as observed in tests, a prestressed concrete structure is likely to suffer greater deformations or be called upon to resist higher forces under most strong earthquakes than a reinforced concrete structure of comparable mass and stiffness. A point in favor of prestressed concrete is that, to resist a given set of forces, a prestressed structure is normally considerably more flexible than its reinforced counterpart. This is a desirable feature for seismic resistance and partly counteracts the effect of the smaller energy dissipation under cyclic loading.

Spencer [46] studied the nonlinear dynamic responses to a strong earthquake (first 8 sec. of the N-S component of the 1940 El Centro record) of two reinforced and six prestressed concrete versions of a 20-story frame structure. An idealized bilinear moment-rotation hysteresis loop for the prestressed members was used. The prestressed structures were found to undergo higher lateral displacements and interstory drifts than the comparable reinforced concrete structures. However, the sectional ductility requirements of the prestressed structures were markedly lower. Studies directed toward confirmation and generalization of Spencer's observations would be most useful.

Further research is also needed in the following areas: damping tests of prestressed concrete structures; high intensity cyclic loading tests of prestressed concrete members and their connections.

CONCLUDING REMARKS

1. Experience in the earthquakes of the last 15 years has shown that both protection of human life and superior damage control can be attained in buildings stiffened by properly proportioned and detailed structural walls.

2. The incompatibility of flexible frames with brittle infills and finishes caused high economic damage to "nonstructural" building components in many earthquakes.

3. The effects of rigid elements on the seismic performance of structures make it imperative that proper account be taken of such elements in design. In reality, there are hardly any nonstructural elements, unless they are deliberately and carefully isolated from the structure itself. All elements attached to the structure and strained during the earthquake participate in the seismic resistance.

The discussion presented so far in this report points to the need for further research in a number of areas:

A. Integrated analytical and experimental research must be carried out in order to lay a basis for the safe and efficient design of the three basic framing systems - frames, structural walls and frame-wall systems.

Dynamic inelastic analyses are needed to arrive at reasonable estimates of the strength (with particular reference to shear) and deformability requirements in critical regions of the framing systems, corresponding to different combinations of significant structural and ground motion parameters. The ground motion parameters of concern are the intensity, the duration and the frequency characteristics of the excitation. The input motions to be used in dynamic analyses should be selected such that, for a particular intensity and duration, the frequency characteristics induce critical (near resonant) excitation in a structure in both the elastic and the post-yield stages of its response. This would normally require the use of a number of input motions in the analysis of the same structure.

Experiments are required to determine the minimum strength (again with particular reference to shear) and deformation capacities available in properly proportioned and detailed segments of the framing systems. The most promising experimental approach at the present time appears to be the testing of actual structures or large-scale models under equivalent pseudo-static forces intended to induce effects similar to those of real dynamic excitations. Particular attention must be paid to the selection of the appropriate combination and sequence of variation of the equivalent pseudo-static forces.

B. Experimental research is needed to establish force-deformation characteristics for all nonstructural elements incorporated into earthquake-resistant buildings, under appropriate repeatedly variable load combinations. On the basis of the data generated, rational limitations must be established on allowable interstory distortions. The allowable limits must be tied in some way to the intensity of the input motion.

Rational limits must also be established on the overall lateral deflection or drift. The prime consideration here is the stability of a structure under the action of gravity loads, when displacements are large.

The above research must be coupled with analytical studies employing dynamic inelastic analyses, leading to results such as those illustrated in Fig. 10. It would then be possible to translate drift and interstory drift limitations into allowable upper limits on the structural fundamental period (or lower limits on the overall flexural stiffness).

C. In order to establish the suitability of prestressed concrete for use in primary resistant structures against earthquakes, research needs to be carried out in the following areas: a) nonlinear seismic analyses of prestressed concrete structures, b) high-intensity cyclic loading tests of prestressed concrete members, their connections and subassemblages, and c) tests to establish differences in damping characteristics between reinforced and prestressed concrete.

D. Research should be directed to developing the full potential of the special devices and mechanisms which seek to reduce the forces induced in a structure by earthquake ground motions. Reliability studies in the form of earthquake simulator tests on large-scale models of these devices must play a prominent role in such research.

The above items are listed in the order of priorities the authors attach to the various research needs.

ACKNOWLEDGMENT

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

SOFT STORY CONCEPT APPLIED AT ST. JOSEPH HEALTH CARE CENTER
Tacoma, Washington

A Summary Paper Presented at the ERCBC Workshop
University of California, Berkeley, California, July 1977

by Alexander Popoff, Jr.
Vice President, ABAM Engineers, Inc.

In the soft story concept a shock-absorbing "soft story" is introduced at the base of the building to isolate it from seismic excitations. Typically the soft story has a bilinear force-displacement characteristic with a rigid response for wind forces and very elastic (soft) response for the stronger seismic forces. In an earthquake, damage is confined to the soft story. The concept was first described by Fintel and Khan in 1969 (1). The first actual application of this concept was in the St. Joseph Hospital. The structure was completed and occupied in 1973.

The structure is comprised of three distinct sections. Figure 1. At the base there is a large two-story building housing the hospital administration and special services. This is waffle slab, beam and column construction with shear walls around the perimeter and at the central core.

The tower, which is 9 stories high, is basically circular in plan, has 15-inch flat plate floors, 10-inch exterior walls and 8-inch interior core walls. The amount of walls and the undulating shape of the exterior walls make the tower very rigid. Figure 2.

Separating the tower from the base unit is a 36-foot high plaza story, the central core of which houses a 2-story mechanical space. There are no shear walls in the plaza section; the tower is supported by sixteen 36-inch diameter spirally reinforced columns and twenty-four columns of various sizes around the elevators and stairs.

This high plaza story affords an ideal place for a shock-absorbing soft story. The stiff portion of the bilinear force-displacement response is provided by the columns. After the columns yield, the energy is absorbed by the diagonal "braces" made of stress-relieved seven wire 270K strands. The yield level and elongation capacity of prestressing strands are ideal for energy absorption. There are 16 sets of "braces," 4 pairs in the N-S direction and 4 pairs in the E-W direction. They are located between the columns surrounding the stairs and elevators in the core. Figures 3 and 4.

DESIGN PROCEDURE

An outline of the design of the energy absorbing soft story is as follows:

1. A code seismic design was carried out and shear force determined for the level at the top of the soft story. $V_u = 1.4$ V. Using this force, the strand in the tendons was designed.

2. The columns were designed for 1.4 (D+L+E) + 25% of the code shear from 1, using $\phi = 0.75$. The yield moment of the columns was found using the column interaction diagrams, for 1.0 (D+L) using $\phi = 1.0$ and the yield shear force was calculated.

3. The effect on the columns of the vertical force multiplied by the horizontal deflection ($P \times \Delta$) was calculated and resolved into a horizontal force.

4. Using the forces and deflections from above, the force/deflection diagrams were drawn for the columns, the tendons and the P-Δ effect. See Figure 5.

5. The single diagrams were then added to produce the force/deflection diagram for the building. See Figure 6.

   This diagram shows that the columns will yield shortly after the forces for a code earthquake have been exceeded. The strands will then carry on absorbing the energy produced. Note that the strands absorb the energy within their elastic range.

DYNAMIC REVIEW

The structure was analyzed for nine different earthquakes: N-S component of 1940 El Centro, and eight simulated earthquakes developed at California Institute of Technology (2). The simulated earthquakes are intended to represent various intensity quakes ranging from a Richter 8 earthquake to a magnitude 5 earthquake.

The structure was idealized as a one degree of freedom system. Figure 7. The upper floors were considered infinitely stiff relative to the first story. The actual bilinear force-displacement curve of the soft story was used in the analysis. Figure 8.

The response of the structure is given in Figures 9 and 10. From these, two conclusions can be drawn:

1. The structure is highly resistant to earthquake motion due to its large capacity of energy absorption.

2. Direct comparison of the maximum response of the structure to earthquakes of different intensities -- El Centro, A-1, A-2, and B-1 -- indicates that the response is insensitive to the intensity of the earthquake due to the large capacity for absorbing energy.

CREDITS

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Structural Engineer:	ABAM Engineers, Inc., Tacoma, Washington
General Contractor:	Baugh Construction Company, Seattle, Washington
Owner:	Sisters of Saint Francis

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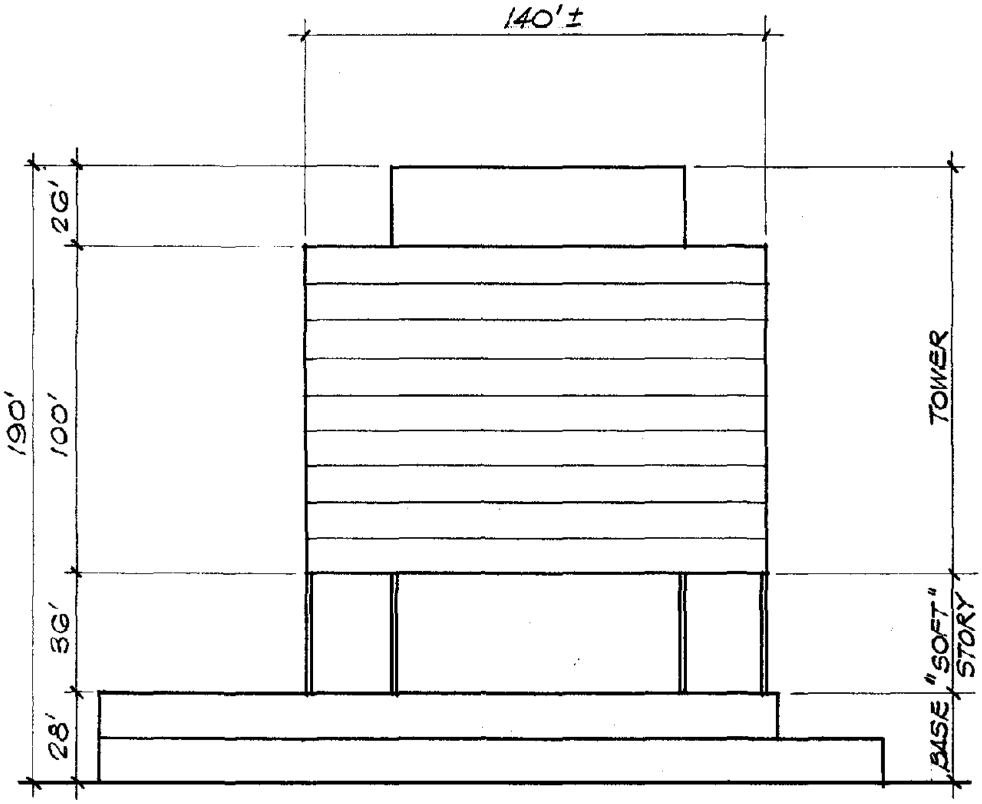
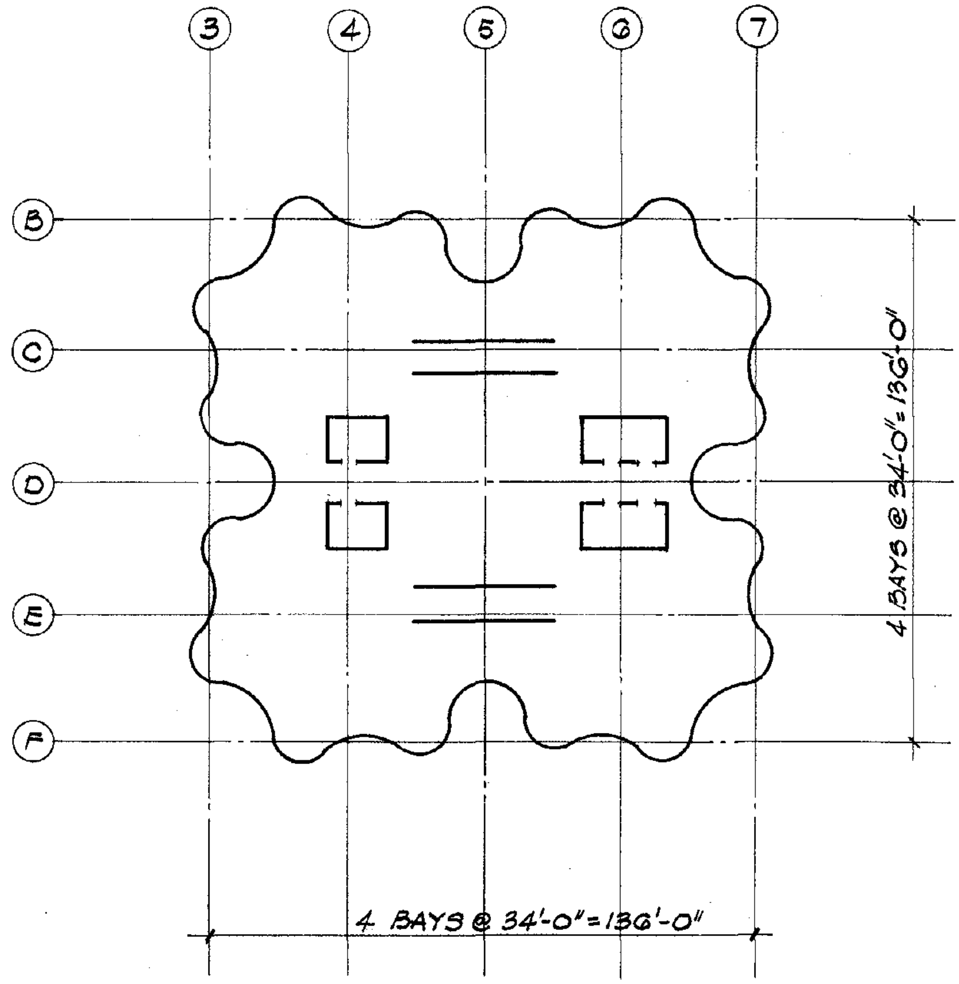


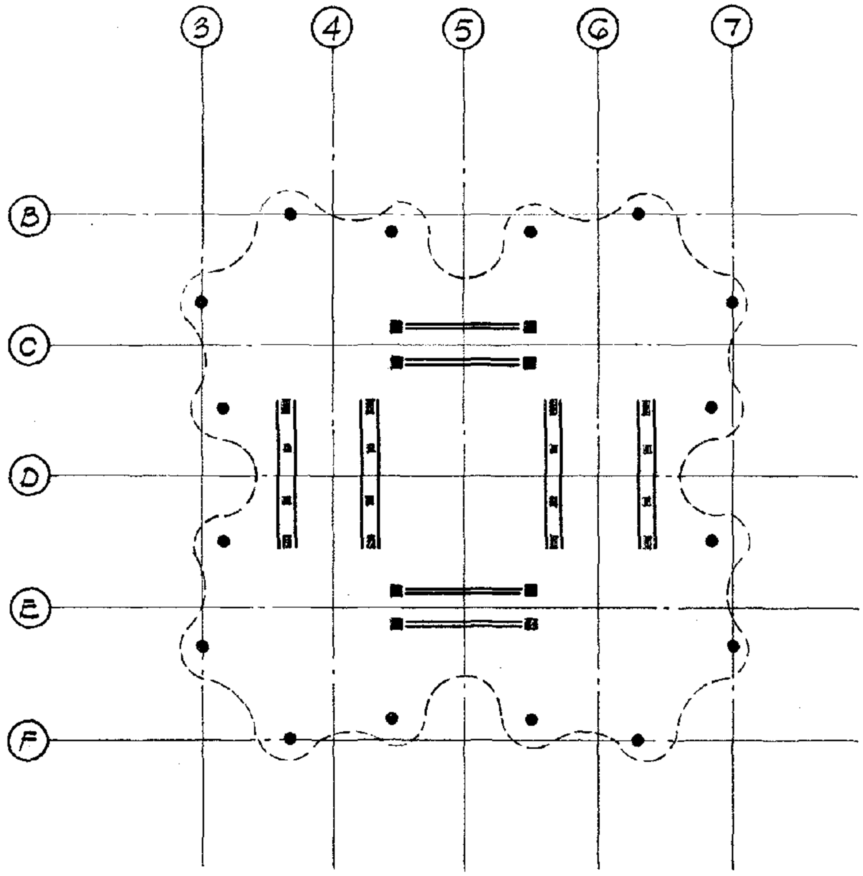
ST. JOSEPH HOSPITAL

FIG. 1



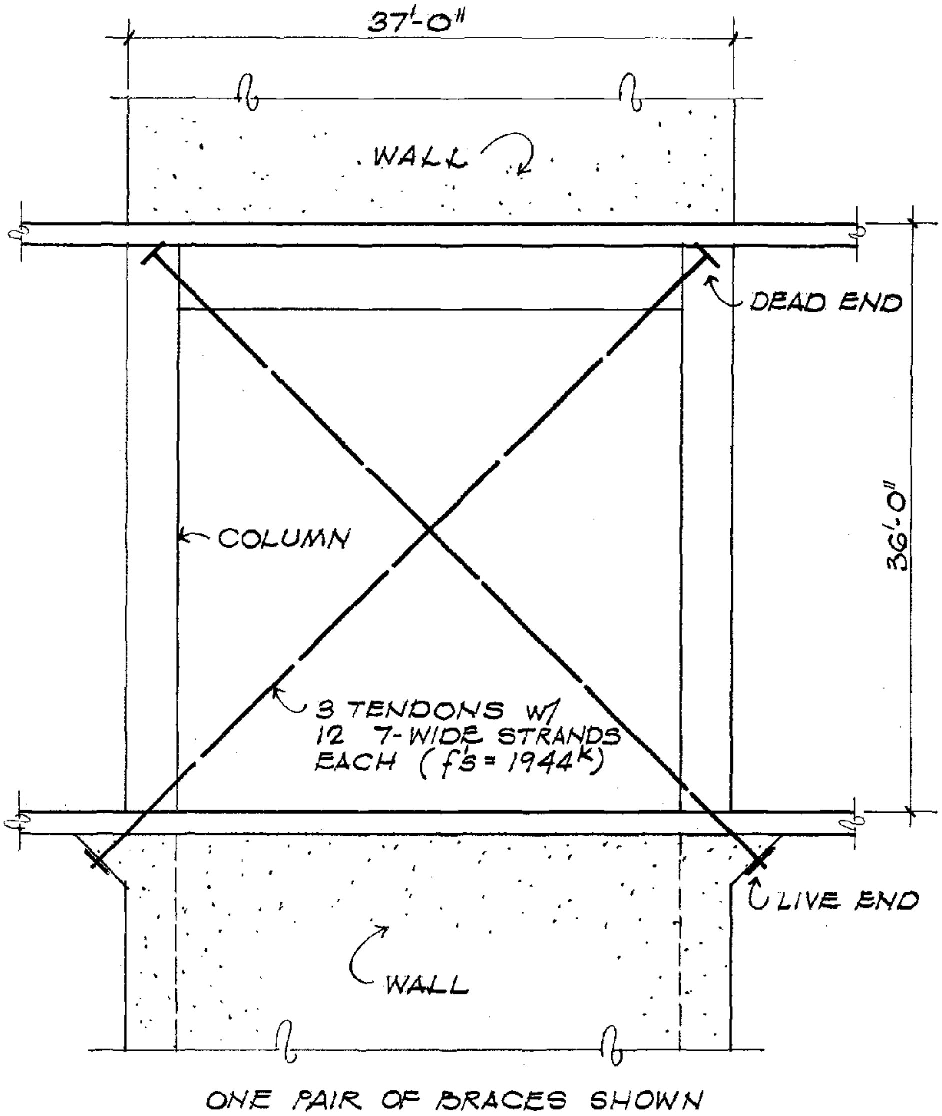
TOWER PLAN

FIG. 2



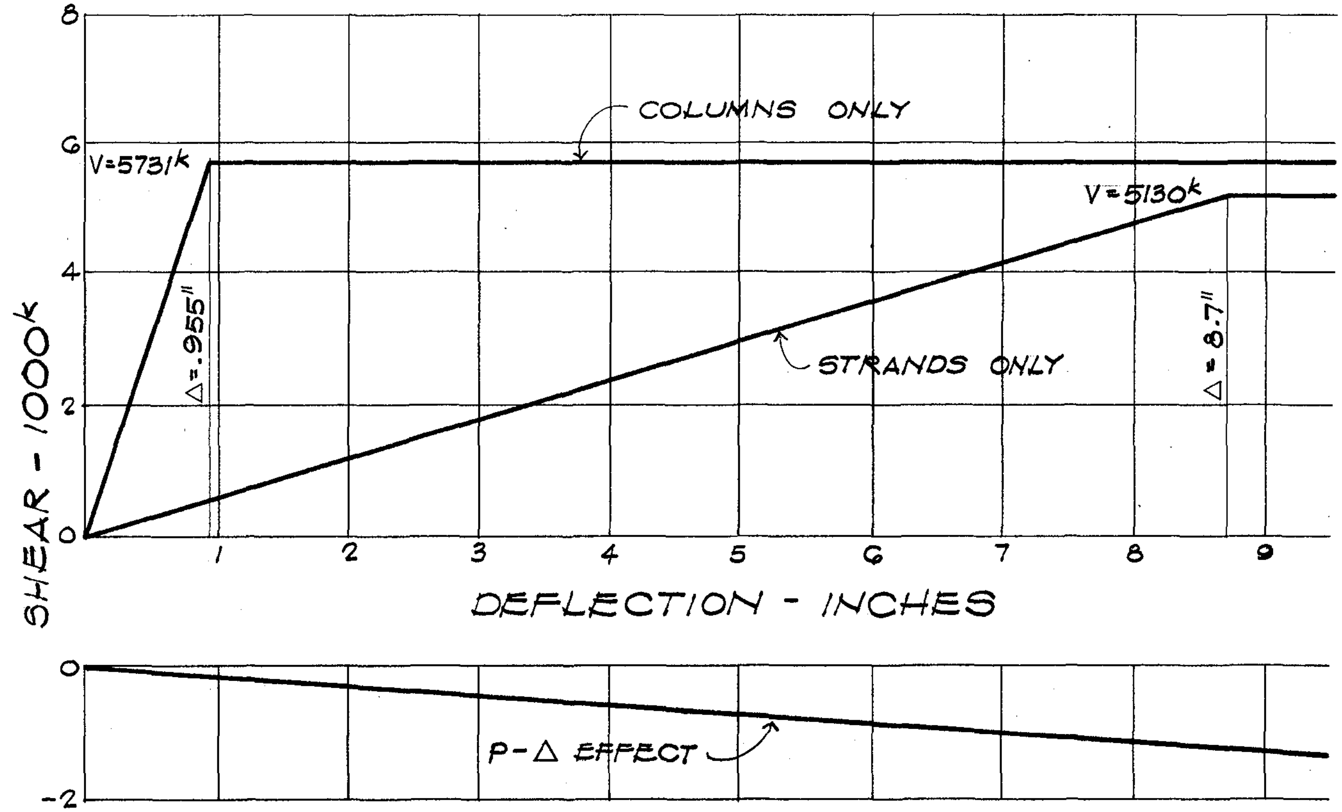
"SOFT" STORY PLAN

FIG. 3



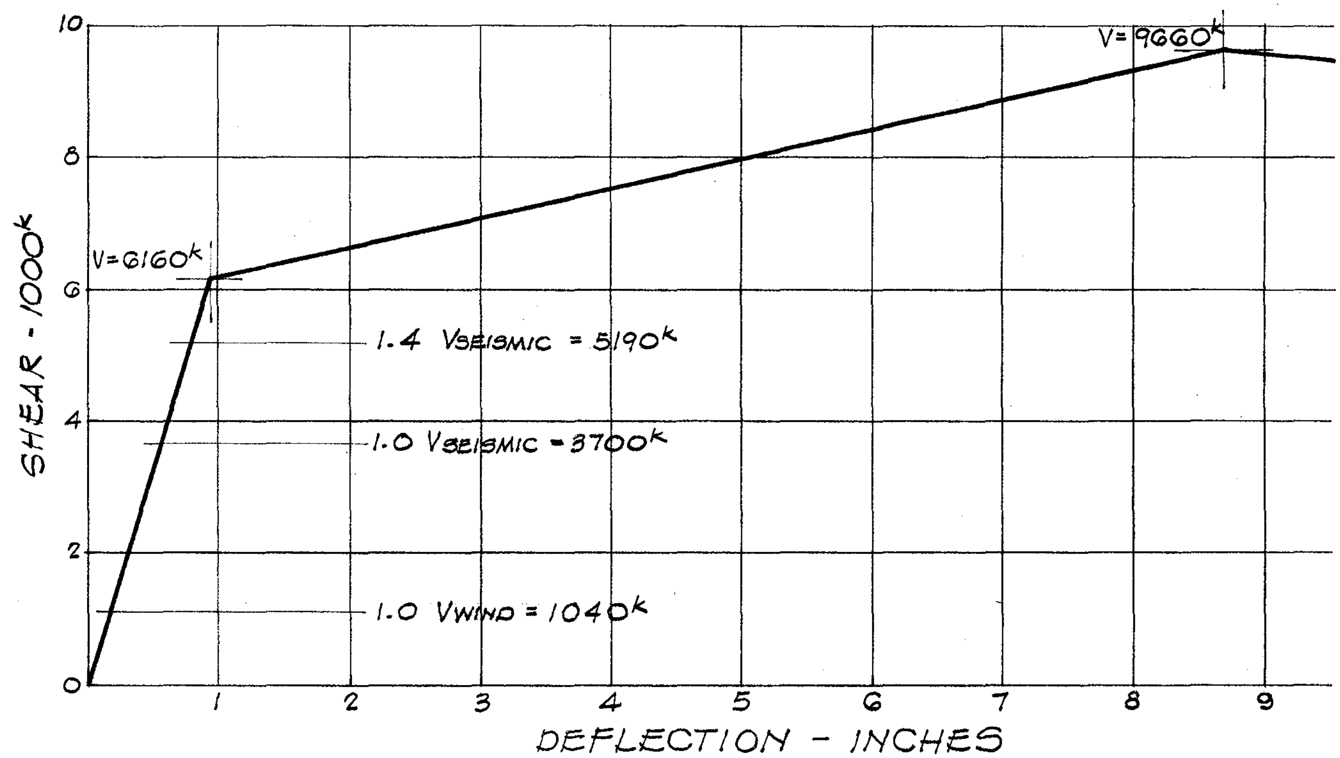
BRACING SYSTEM

FIG. 4



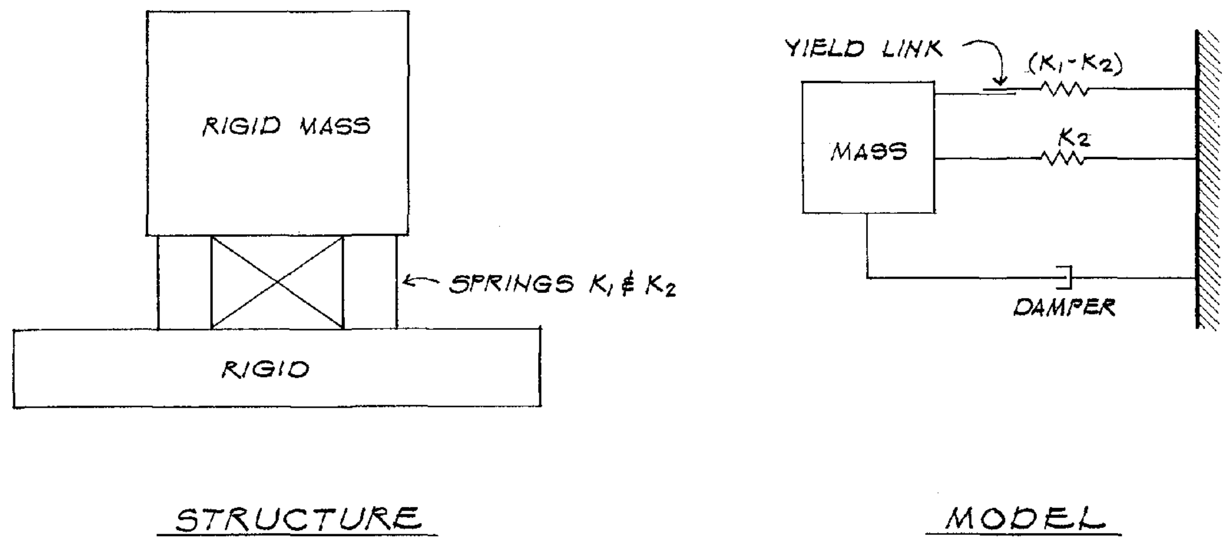
STIFFNESSES OF STRUCTURAL ELEMENTS

FIG. 5



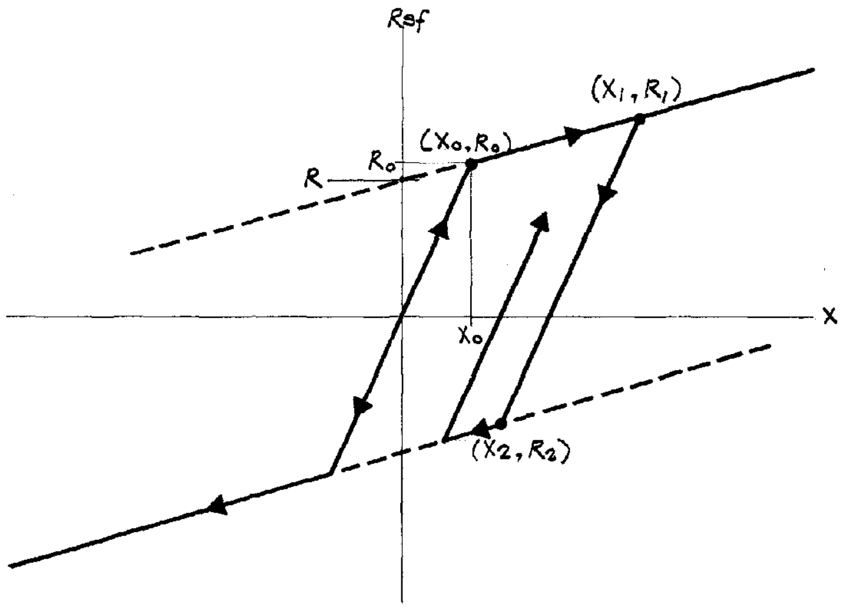
V - Δ RESPONSE OF STRUCTURE

FIG. 6



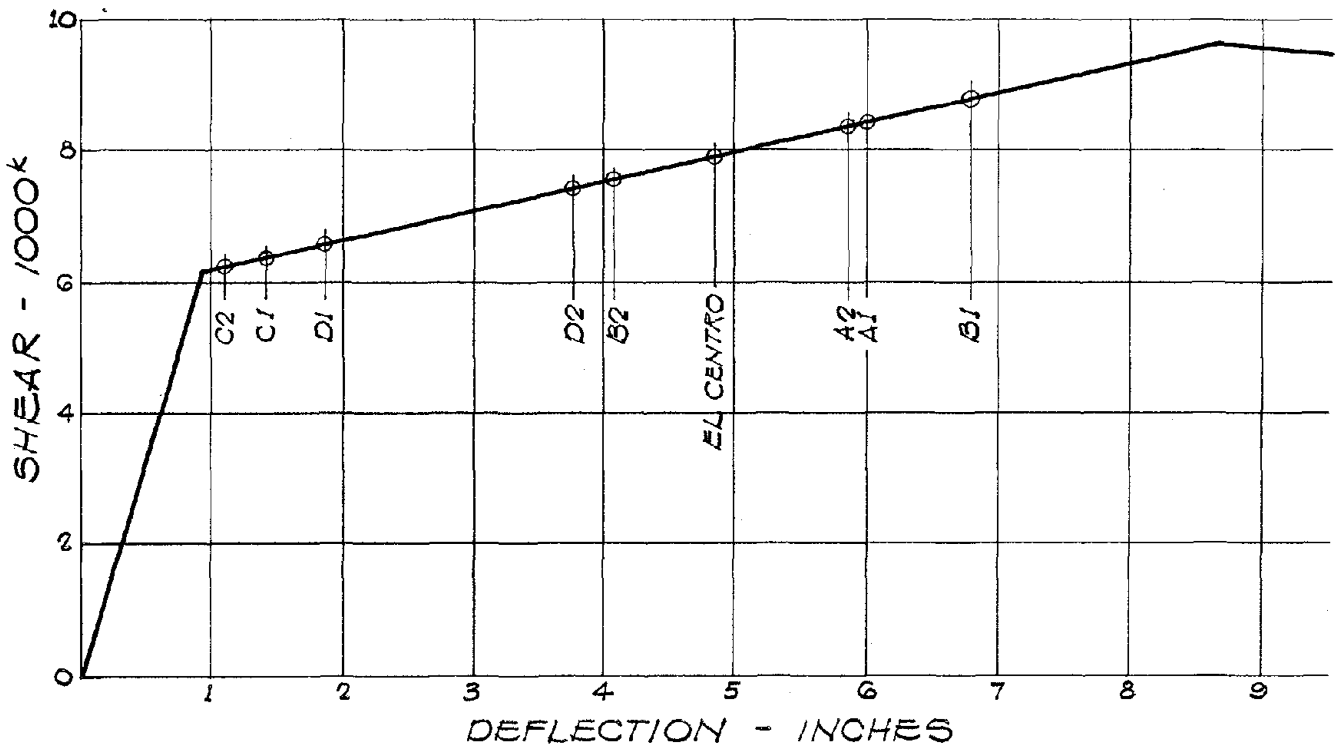
DYNAMIC IDEALIZATION

FIG. 7



BILINEAR FORCE - DISPLACEMENT

FIG. 8



DYNAMIC RESPONSE

FIG. 9

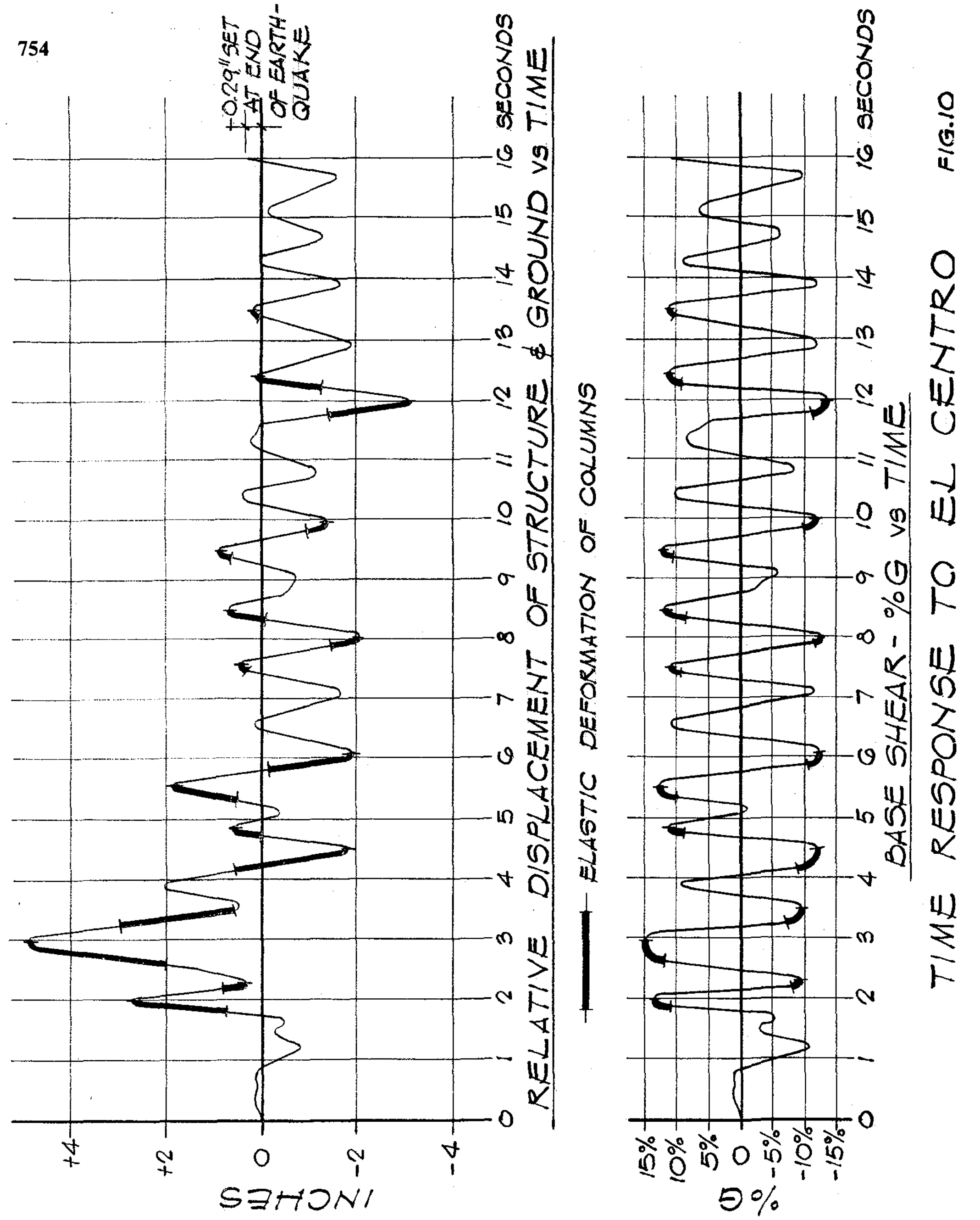
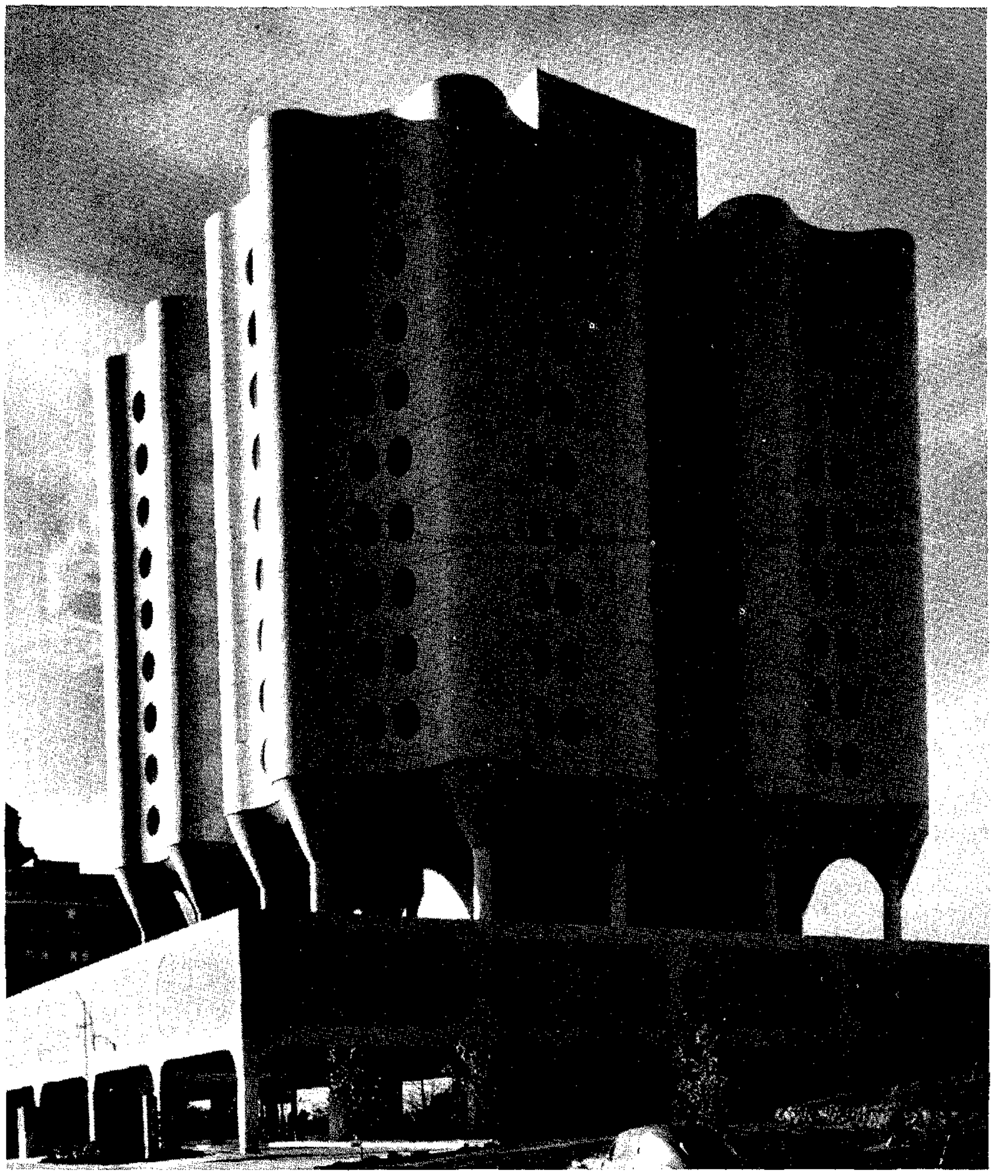

RELATIVE DISPLACEMENT OF STRUCTURE & GROUND vs TIME
INCHES
+4
+2
0
-2
-4
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 SECONDS
+0.29" SET AT END OF EARTHQUAKE
ELASTIC DEFORMATION OF COLUMNS
BASE SHEAR - %G vs TIME
G %
15%
10%
5%
0
-5%
-10%
-15%
TIME RESPONSE TO EL CENTRO

FIG.10



St. JOSEPH HEALTH CARE CENTER

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15. 1977

THE 18-STORIED SHIINAMACHI BUILDING

Cast-in-Field Reinforced Concrete Systems
discussed by

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INTRODUCTION

Both Tokachi-Oki(1968 Japan) and San Fernando (1971 U.S.A.) Earthquakes revealed the defect of reinforced concrete structures. The earthquake resistant capacity must be estimated by the combination of strength and ductility in DMRF (Ductile Moment Resisting Frame). These two factors are introduced in "The Aseismic Capacity Evaluation of Existing Low-Rise Reinforced Concrete Buildings"[1] and in "Newly Proposed Earthquake Resistant Design"[2] issued from the Ministry of Construction of Japan, 1977.

It was discovered after the Tokachi-Oki Earthquake that RC columns whose lateral reinforcements consist of only hoops did not maintain as much efficient deformability and ductility as those with spirals or additional crossties did. The fact is due to the difference of confinement effect of core concrete and longitudinal bars of columns.

This paper deals with the test results of columns with effective transverse reinforcement arrangements which enabled the design and construction of high-rise concrete building (18 stories) for the first time in Japan.

REVIEW OF THE RECENT WORKS

Soon after the San Fernando Earthquake, H.Aoyama et al [3] tested spiral columns and showed that the spiral reinforcement was very effective in acting as confinement and web reinforcement, and that the column owed its success to confinement even though the confined core concrete had completely crumbled.

The most eminent research works in the field of how to improve the ductility of reinforced concrete columns are the national projects in Japan [4] carried out by the committee sponsored by the Building Research Institute of Construction Ministry in cooperation with many universities and research institutes of construction companies.

The failure modes and factors which affect ductility under cyclic loads are studied and the following conclusion is derived from 125 short columns tests. As the factors which control ductility of columns, buckling of compression reinforcements, shear failure and bond failure are considered important. To prevent compression reinforciments buckling until large deflection of column, it will be effective to keep spacing of web reinforcements less than 8 times diameter of the axial reinforcements. There are certain conditions for the combination of tensile reinforcement ratio ρ, compressive stress σ and shear arm to depth ratio Mc/QD where shear compression failure cannot be avoided even with much web reinforcements. This condition could not be made so apparent, but Eq.(8) is considered to be one approach. It seems effective to keep the flexure capacity within about 1.4 times the initial bond-split cracking load shown by Eq.(8). It is not effective to increase rectangular type hoops but it will be effective to put spiral hoops closely.

$$\frac{\sigma_1}{{}_c\sigma_t} + 6\,\frac{X_n}{D} \leqq 3 \tag{8}$$

where, σ_1 : tensile principal stress
$_c\sigma_t$: tensile strength of concrete
X_n : distance from neutral axis to compression fiber
D : column depth

From the most recent paper of the same committee it is reported [5] that columns whose tensile reinforcement ratio is less than 0.6 % showed ductile behavior. Those columns of welded hoop or rectangular spiral hoop with subties are also ductile even under high axial stress ($\sigma/f_c' = 0.33$). And that the effective transverse reinforcement arrangement improved the ductility of short columns (M/QD=1) with the same amount of shear reinforcement ratio. Supplementary ties with welded or spiral hoops which are continuous and closed help very much in improving ductility, and at the same time they contribute to preventing bond-splitting along tensile bars.

The double spiral hoop specimens were studied by T. Shimazu [6] and it is reported from 26 short columns tests that the maximum load of the double spiral column is much higher than that of conventional hooped columns. The hysteresis loop of load-deflection curve, too, is much more stabilized even at large deflections.

RESEARCHES AND DEVELOPMENTS IN KICT
(Kajima Institute of Construction Technology)

As is a matter of common practice in Japan, buildings with pure reinforced concrete (RC) structure are not regarded as safe enough against earthquake forces as those with steel (S) or steel composite reinforced concrete (SRC) structure. In order to construct a rational high-rise building with pure RC the KICT had to conquer many disadvantages that lied in the design and field work of reinforced concrete structure. Therefore, it was necessary to establish aseismic criteria for the structural design. The safety of the designed structure had to be confirmed with full accuracy both by structural experiments and dynamic analysis. The former part of this chapter discusses the experimental findings on aseismic RC structures, whereas the latter part reports on the earthquake resistant design of the 18 storied Shiinamachi Building which is the tallest RC structure in Japan.

Experimental Findings

Comparison of Transverse Reinforcement Types--As the first step, columns with three types of lateral reinforcements as shown in Fig. 1 were tested with three levels of reinforcement ratio [7]. The difference of ductility of these columns was obvious and their restoring force ratio is shown in Fig. 2 under cyclic reversals at the deflection angle of 1/100. Thus the hooped columns lose their load-bearing capacity after a few cyclic loadings. On the other hand tied or spiral columns maintain their capacity.

Newly Developed Transverse Reinforcements--Although structural experiments testified to the efficiency of tied columns, they have a few difficulties in time of construction. It is, therefore, hoped to look for rational and more workable reinforcements for columns, and the combination type of spiral and hoop reinforcements was developed [8]. This reinforcement arrangement was named as KS type. It showed ductile behavior similar to tied column. It was just after these test had been completed when the quake hit San Fernando in California. Then, KS type columns were tested in the same procedure and in the same testing apparatus as the national projects of Japan [4] in order to be compared with other reinforcement types as shown in Fig. 3. And here again KS type columns showed the most ductile behavior as in Fig. 4, and the next were T type (subtie) columns and spiral columns under the same reinforcement ratio [9],[10].

Beam-Column Joints (Exterior Column)--Among many construction works of the RC structure, reinforcing bar assembling is one of the most predominant that governs the whole construction periods. It was, therefore, proposed to assemble previously the reinforcing bars on the ground. So it is eventually indispensable to use simplified details of the beam-column joints including anchorage of beam bars. Four types of anchorage system for beam-column joint of outer column were tested as shown in Figs. 5 and 6. The test results indicate that newly developed anchor types can assure strength, stiffness and ductility enough to be compared with the conventional type as shown in Fig.7 [11].

Reinforcement at Beam-Column Joint (Interior Column)--In order to avoid the complex assemblage of reinforcing bars at beam-column joint portion, it was necessary to look for an adequate and suitable shear reinforcement. If simpler and sparser arrangement of steel bars were proved enough to sustain earthquake forces, a large amount of labour would be saved.

Four types of shear reinforcement arrangement applicable to beam-column joint portion were tested as shown in Figs. 8 and 9. Here, unlike in the case of column specimen, 'A' type (hoop alone) could sustain quake forces up to 2/100 deflection angles or more. It was thought that this behavior owed to the confinement effect by re-bars and concrete of beams jointing transversely at the joint.

Splices of Large Size Re-Bars--The use of large size re-bars can simplify the structural details and make the labour work easier. Splices of large size bars over 35 mm in diameter were not achieved yet in Japan, although they are widely used in the world. Some tests of Cadweld joint were performed and the the official approval of building bureau of Japan was obtained.

Structural Design of Shiinamachi Apartment House

Aseismic Desigh Criteria--Basic criteria of earthquake resistant design were established classifying earthquake intensities into three classes, and regulating the response of the building or degree of damage as follows:

Class I	Moderate earthquake (approx. 0.1G)	---	No structural damages
Class II	Severe earthquake (approx. 0.3G)	---	No parts of building structure reach yield stress level
Class III	Worst earthquake (approx. 0.5G)	---	Slight entry into the plastic range is allowed but the structural members never collapse

Outline of the Building--The building is of 18 stories, 49 meters high, 3 spans for transverse and 8 spans for longitudinal framing with typical floor of 13.5 x 24 square meters as shown in Fig. 10 (a). The building structure is designed to be composed of open frames. Additional shear wall is arranged at the basement.

Columns are all 60 cm square of KS type shear reinforcement whose ratio ranged from 0.75 to 1.20 % determined from the various experiments as in Fig.

10 (c). Beams are 60 cm in depth with 35, 40 and 45 cm in width as in Fig. 10 (d). A stirrup tie used for longitudinal beam is unprecedentedly to be formed by placing U shaped reinforcing bar. After placing bottom longitudinal reinforcements on the U-bars a reinforcement cage of transverse beam is set as shown in Fig. 10 (e), and the top longitudinal bars of longitudinal beam are placed upon it. Main re-bars of longitudinal beams are anchored by developed plate anchorages as shown in Fig. 10 (f).

The stirrup tie for transverse beam is shaped into conventional closed type in the preassembling stage on the ground and the anchorage of main re-bars are continuous U-anchorage type. To give ductility, it is indispensable that shear strength of each member surpasses the binding one. Yield shear force of each story is, therefore, determined as the sum of column shear transformed from the bending moment. The yield shear coefficient at the first floor level is found to be 0.35 after nonlinear frame analysis.

Post-tension pre-stressing is introduced in high strength steel placed at the center of outer columns below 5th floor as denoted PC-BAR in Fig. 10 (b) in order to reduce the tensile cracking stress of column concrete by overturning moment, although the resistance to quake shear force is preserved by the rest of the columns.

Safety Confirmation by Structural Experiments--As the project was the first of this kind in Japan in which a tall building was constructed with pure RC structure, several structural experiments were performed. Those included 2 series of tests on the columns and a test on the interior framing subassemblage.

Through former column tests, the effect of Cadweld joint in a column was comparatively investigared with the unspliced column. Effects of pre-stressing of outer columns were also observed and load-bearing capacity of column subjected to tensile force was proved to maintain to some extent without significant failure. Test specimen and one of the test results are shown in Fig. 11. Here in the test procefure, the axial force N was proportionally changed to the horizontal shear force V (N/V=11, V=P/2, P:jack load).

Fig. 12 shows an outline of the test on the subassemblage specimen picked out of longitudinal interior framing at fourth story level. By applying load to the beam as well as constant axial load to the column, earthquake forces are produced in each member of the subassemblage. Loading to the beam was alternately repeated ten times at the distortion of 1/100 deflection angle of column, then gradually increased up to 5/100. From the deflection curve, a decrease of load bearing capacity after 10 cucles of loading with story drift of 1/100 deflection angle is only 15 % or less. At the maxumum loading level defined by ultimate moment of beam, strain of shear reinforcing bar is half of the yield limit. It is observed that the test specimen has sufficient ductility under large story drift of 5/100, and that there are no structural defects along the construction joint.

Safety Confirmation by Earthquake Response Analysis--Dealing with a complicated dynamic behavior of the building structure accompanied by cracking and yielding, and idealized vibration model with lumped mass is established. As shown in Fig. 13, the vibration model is assumed to have two kinds of stiffness. One is shearing stiffness which is concerned with each deflection of framing members, while the other is bending stiffness which is due to total or whole bending of the framing. Shear stiffness is calculated and assumed to have nonlinear degraing property as shown in Fig. 14. Bending deformation is much smaller than shearing one, and the stiffness is assumed to remain elastic.

Supposing the building structure is fixed on the first floor, fundamental vibration periods in fully elastic range are 0.81 and 0.95 seconds in longitudinal and in transverse direction respectively. In accordance with the aseismic criteria, maximum response values are studied by several input earthquakes such as El Centro 1940 NS, Taft 1952 EW, Tokyo 1956 NS and Sendai 1962 NS whose maximum intensities of ground motion are 0.1, 0.3 and 0.5 G. Fig. 15 shows one of these response data.

Innovation in the Construction--Two biggest problems, time and quality, were to be solved before the actual execution of construction of high-rise reinforced concrete building. A new method of prefabricating and assembling reinforcing bars on the ground was introduced to save time. And their joints were innovated together with lap and gas-welding joint as well as Cadweld method. Also, the formwork was developed by using large panels. Each of the large formwork units covers an area of 12 x 3.4 m^2 and weighs approximately 3.5 tons.

Concrete strength was confirmed for the purpose of three objects in the testing laboratory set up on the site.

a) Confirmation of the strength for the formworks removal
b) Assurance of designed strength of structural concrete
c) Quality control of fresh concrete

The formworks are decided to be removed or not after a compressive strength test has been finished. Testing many cylindrical specimens, the structural strength of each member was also confirmed. Results of the testing showed least fractuation in desired properties.

Summary--As the starting point to an approach of earthquake resistant design of a tall building with pure RC structure, large scale specimens were tested in order to look for improved methods for aseismic members. It was recognized that the available ductility is controlled not only by ratio of shear reinforcement but also their placing types. Thus, the RC structure was successfully designed which deserves the name of DUCTILE MOMENT RESISTING SPACE FRAME.

The maximum ductility requirement against the most dangerous earthquake with 0.5 G was less than 1.7, which means that the aseismic quality of the building far exceeds seismec criteria. It is concluded, therefore, that the design procedures of this 18 story building lead to rather conservative results. These results are, however, attributed to the fact that this was the first time in Japan in which a tall building was constructed with pure RC structure.

ADVANCED INTERESTS IN REINFORCED CONCRETE STRUCTURES

Findings from Studies on Slitted Shear Walls [11]

Slitted wall has vertical slits at certain intervals at mid-height of a wall as shown in Fig. 16 (a). These slits are complete breaks not only in concrete but also in reinforcements so as to change the shear wall to a series of flexural wall-columns.

When the slitted wall is subjected to earthquake forces and story displacement, and when its sway deflection becomes large, fine tensile cracks become noticeable at each slit end as shown in Fig. 16 (b). Thus, the slitted wall tends to swell with crack development in plastic range, by which the wall-columns of slitted walls are confined in two directions--horizontally and vertically.

The change of vertical restraining stress σ_n with increasing average shear stress $\bar{\tau}$ is analyzed by FEM and is shown in Fig. 16 (c) altogether with the experimental results. This vertical stress σ_n is proportional to the L/D ratio as in Eq.(1), L and D being the length and depth of a wall-column respectively.

$$\sigma_n = q.\frac{L}{D}.\bar{\tau} \tag{1}$$

Eq.(1) suggests that the additional confining stress σ_n is large in such short columns as in wall-columns of slitted wall.

Findings from RC Short Columns Tests

Many reinforced concrete short columns were tested by the National Project Committee in Japan [4]. The author's group found interests in the confining effect by lateral reinforcements. It is quite appropriate to consider that if the confinement by lateral reinforcing bars is sufficient and effective the strain in them may increase in plastic range. And that a large amount of shortening in column length may occur when the confinement is poor, because the core concrete swells horizontally after cracking.

From the analysis of the shortening phenomenon of columns it is concluded that if the more confinement is given by bar cages or baskets, the larger ductility may be obtained with less shortening of columns. The confining effect derived from strain measurements in lateral reinforcements of 5B Series in Fig. 4 is shown in Fig. 17, where k is the confining coefficient given by Eq.(2) [5].

$$k = A(\varepsilon_{swell}.E_s.A_s)/(V\cdot S) \tag{2}$$

where, A is the constant given by the lateral reinforceing arrangement types, V shear force, and ε_{swell}, E_s, A_s and s are the strain, Young's Modulus of elasticity, cross-sectional area and spacing of lateral reinforcement, respectively.

RECOMMENDATIONS FOR FUTURE STUDIES

(1) Design Procedure of Lateral Reinforcements of Columns

New design procedure of lateral reinforcements in ductile columns should be established taking into account of the confining basket effect by them. To arrive at the final objective, confinement in inelastic range should be studied on their arrangement types, compression intensities, shear arm to depth ratio, and reinforcement ratio etc.

(2) Studies on the Change of Compressive Stress Induced by Earthquakes

From the studies so far conducted by KICT it may be anticipated that the collapse of reinforced concrete short columns are due not only to their small shear arm to depth ratio but also to a larger additional compression which may be induced in them after cracking by sway deflection, because the upward swelling of columns is restrained by the weight of the building. Such a dynamic force to raise the upper floors up by sway deflection might be very large.

This additional compression will increase the flexural ultimate strength and may surpass the shear capacity of short columns in low-rise buildings. So the up and down movement of each floor level should be studied from dynamic simulated tests of short columns structure.

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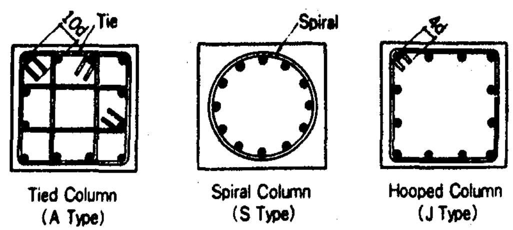


Fig. 1 Typical Lateral Reinforcement Systems of Columns

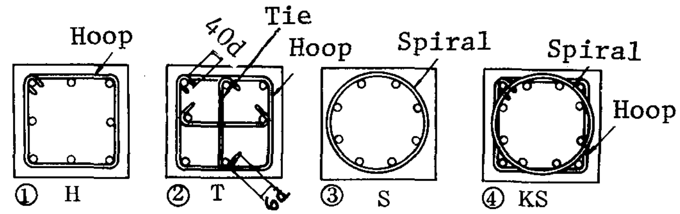


Fig. 3 Various Transverse Reinforcement Types Tested in the Same Apparatus of National Project of Japan

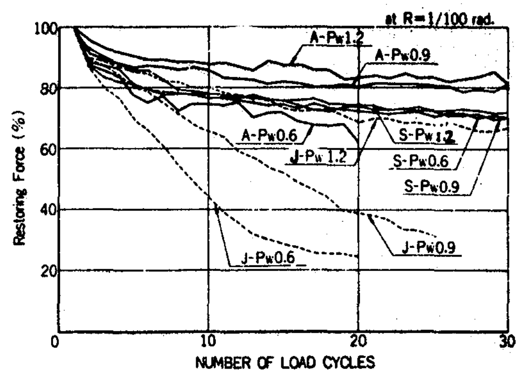


Fig. 2 Reduction of Restoring Force of Columns under Alternately Repeated Distortion

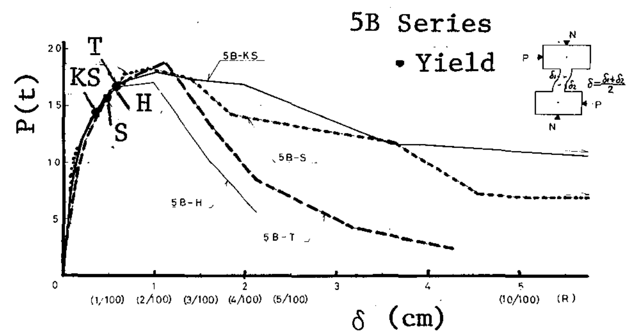


Fig. 4 Enveloped Load-Deflection Curves of Four Types

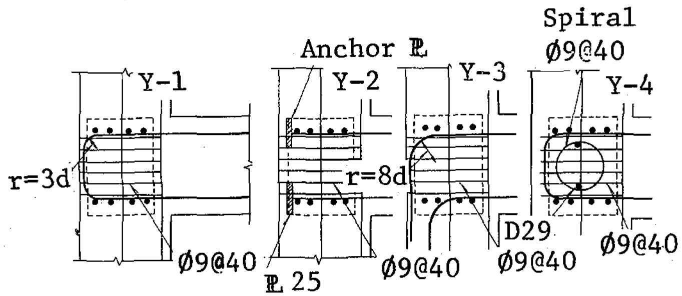


Fig. 5 Various Anchorage Systems of Beam Bars

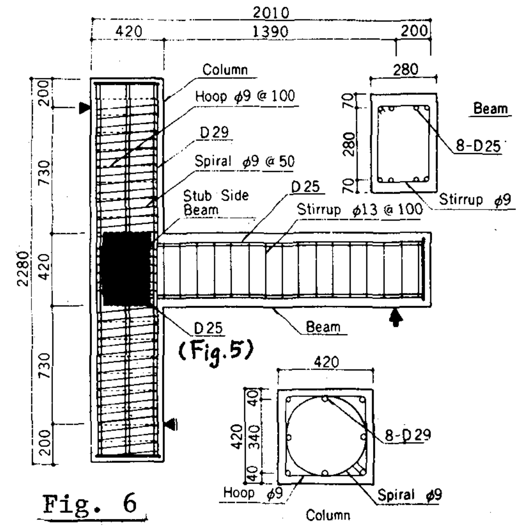


Fig. 6 Loading Method to Beam-Column Joint

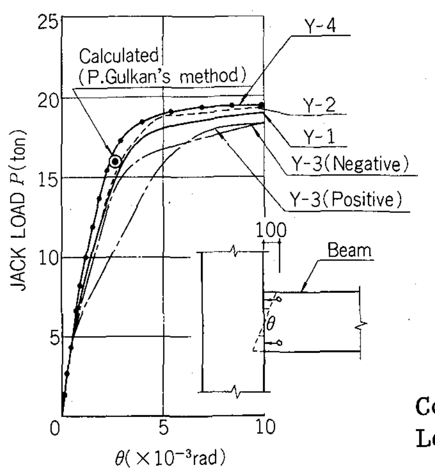


Fig. 7 Comparison of Enveloped Load-Rotation Curves at Beam End

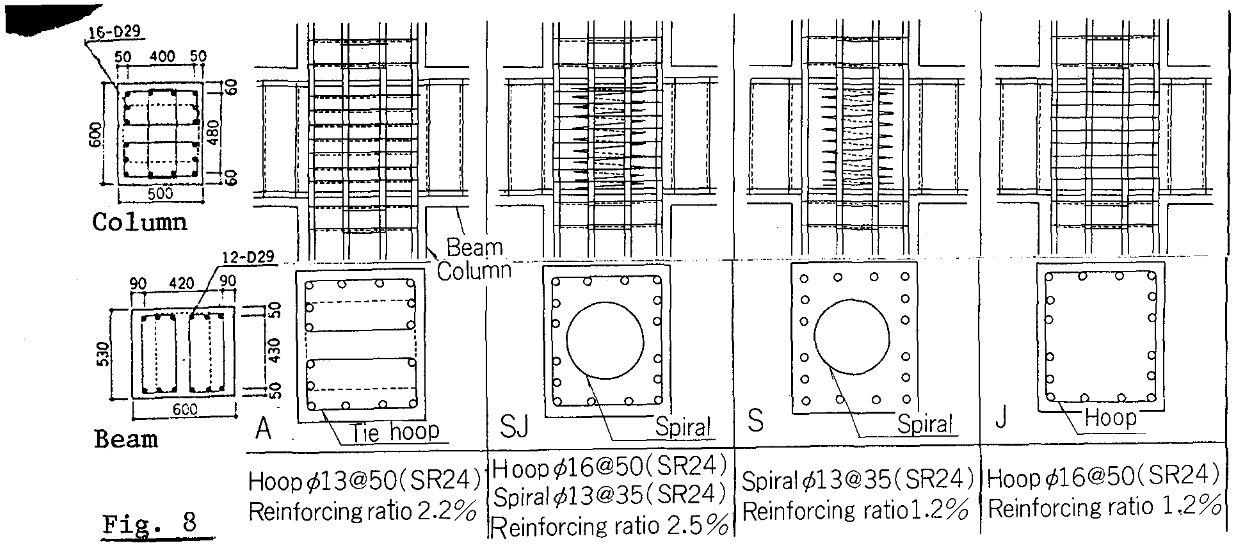


Fig. 8
Details of Beam-Column Joints

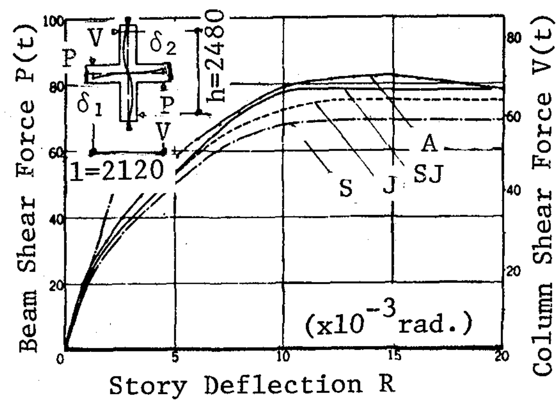


Fig. 9 Comparison of Enveloped Load-Deflection Curves

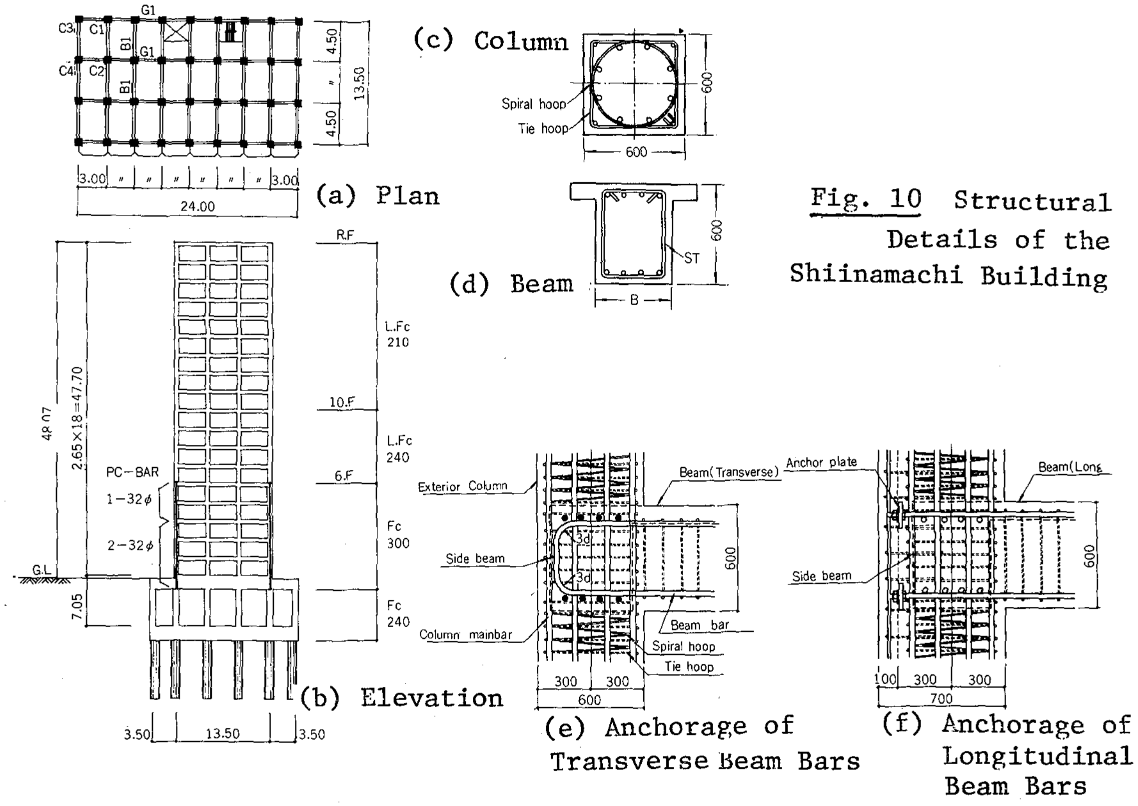


Fig. 10 Structural Details of the Shiinamachi Building

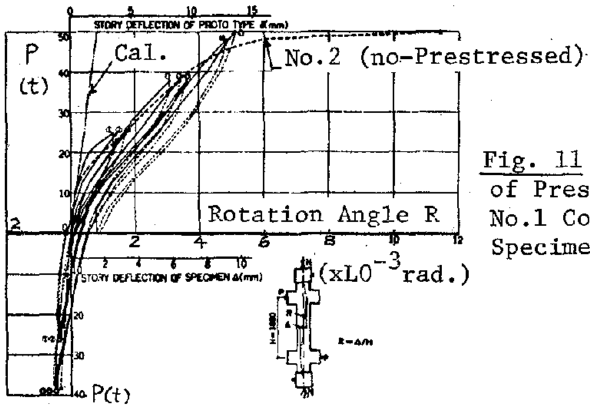


Fig. 11 Load-Deflection Curve of Prestressed Exterior Column No.1 Compared with No-Prestressed Specimen No.2

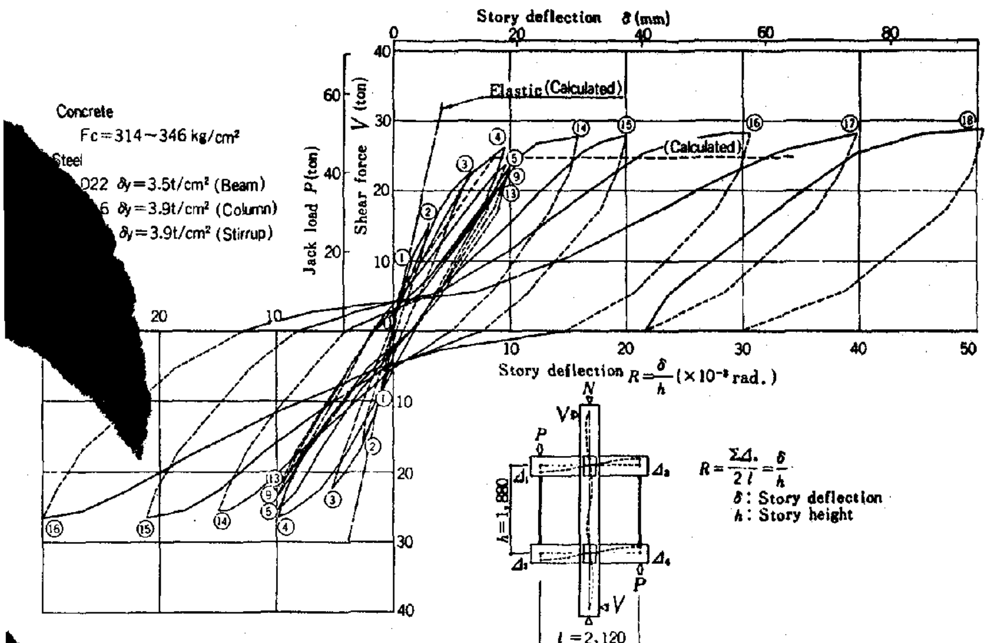


Fig. 12 Load-Deflection Curve of Interior Subassemblage

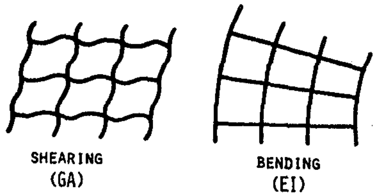


Fig. 13 Vibration Model

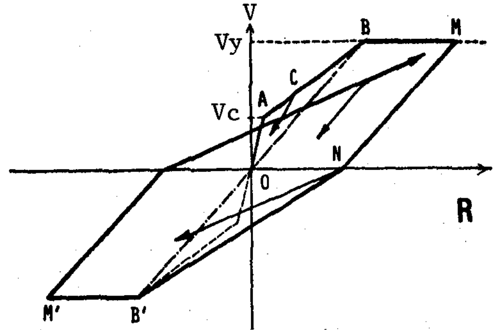
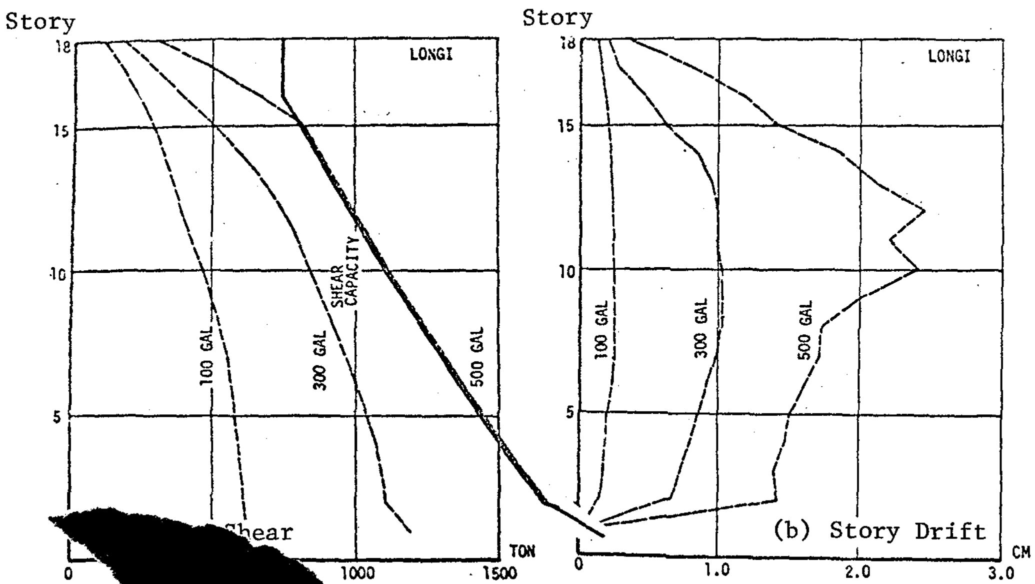


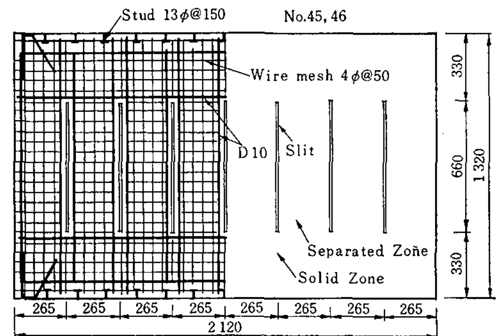
Fig. 14 Idealized Hysteresis Loop (Shear)



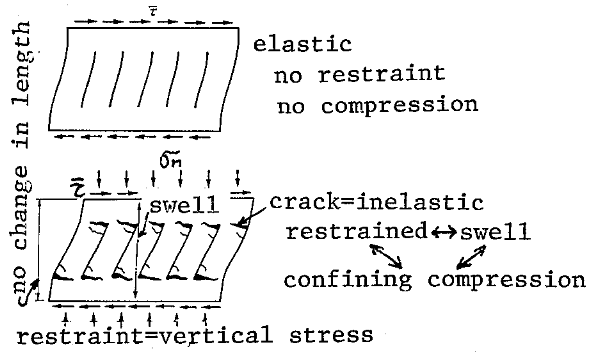
hear

(b) Story Drift

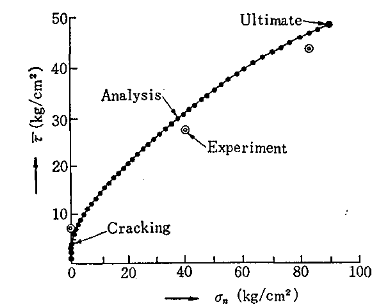
Maximum Response by Taft 1952 EW



(a) Composition of Slitted Wall



(b) Swelling Model



(c) Vertical Restraint by FEM Analysis and Experiment

Fig. 16 Studies on Slitted Shear Walls

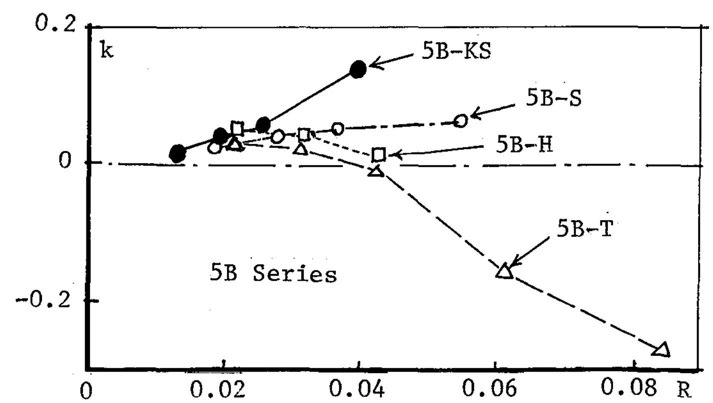


Fig. 17 Relationship between Confining Coefficient k and Deflection Angle R

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

STATE OF THE ART OF PRECAST CONCRETE TECHNIQUE IN JAPAN

by

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SYNOPSIS

This report introduces the state of the art of precast concrete technique in Japan. In the first part the history of precast concrete structure is described from the beginning in 1950's up to date with social and economic background. In the second part is introduced a new building system developed by the authors which can be called "Composite Structure". It is composed of precast concrete unit elements and cast in-situ parts, and at the same time it is reinforced concrete in the longitudinal direction and steel reinforced concrete in the transverse direction. In concluding remarks specific problems yet to be investigated are lined-up and the future style of the effective usage of precast concrete technique is commented though it may not be world-wide applicable.

THE HISTORY OF PRECAST CONCRETE STRUCTURE IN JAPAN

To make clear the technical movement of precast concrete structure, it is necessary to understand its history in relation to the social and economic background. As is well known, Japan has extremely high density of population. At present her total area is 370,000 km^2, 70% of which is mountain area, and her total population is about 113 million. Most of the population concentrates in big cities, so the population density of urban area is very high: for example more than 1,000 persons per hectare (10,000 m^2) in some wards of Tokyo.

To accomodate such a large and highly dense population a large number of housings should be supplied. Prefabricated structure has been expected and utilized to satisfy the needs based on its capability of mass production. The prefabricated structure in Japan has made technical progress from low-rise housing to high-rise one and its quality has been improved, attaining the technical requirements of earthquake resistance and fireproofness which are indispensable for the structure built in high seismicity zone.

Development of Low-rise Prefabricated Housings

One of the most urgently needed works to be done in the destroyed country after World War II is to build a large number of housings. It was necessary to adopt the industrialized method instead of manufacturing method in production of housings to meet the needs. The first prefabricated housing was made of standardized wooden panels and its was called "PREMOS". More than 1,000 PREMOS's were built, but cheaper and lower-quality housings built by conventional method prevailed. Hereafter housings built of factory-made concrete blocks came into vogue on account of its easiness of handling, though they are not necessarily satisfactory in their earthquake resistant capability and waterproofness.

In 1955 Japan Housing corporation (J.H.C.) was founded with the object to supply enough housings and housing lots to laborers suffering from the want of housing. Studies were conducted systematically to build fireproof multiple housings more cheaply, more quickly and with higher quality. In the following decade industrialization of construction was carried out by J.H.C. in cooperation with Building Research Institute of Ministry of Construction and companies like TAISEI CRPORATION. The developed methods were light-gauged steel structure and precast concrete walled structure, both of which were built in trial in the form of two storied terrace-house as shown in Fig.1. The building method of the precast concrete walled structure is called "Tilt-up Method" generally, and the structure is built by tilting-up and jointing precast concrete panels on site. The most important technical key-point of this method was to establish the structural reliability and good workability of joints between precast concrete panels as well as the quality and cost control of precast panels. For this purpose various kinds of experiments were carried out to examine the behavior of horizontal and vertical joints under loading. The joint method in early stage was wet joint both in horizontal and vertical section, casting concrete or mortar into joints after setting up panels. Dynamic tests were also carried out vibrating a full-scale two storied specimen by large vibration-generator to ascertain the structural characteristics of the building as a whole. In 1958 several hundreds of multiple housings were built in the suburbs of Tokyo as the trial of mass production of low-rise precast concrete housings, and research was done on the technical and economic effects. On the other hand research and development was conducted on independent housings.

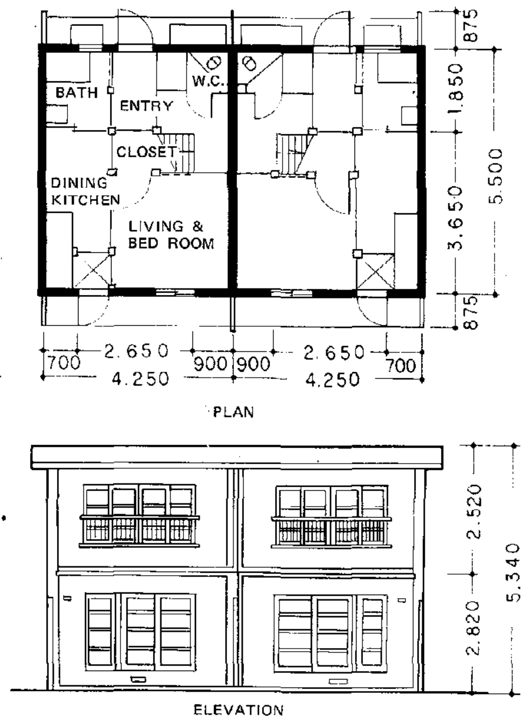

Fig.1 Two Storied Apartment House

Such technical results were edited into "Recommendations for the Design of Special Concrete Structures" issued from Architectural Institute of Japan (A.I.J.).

Extension to Middle-rise Housings

In 1960's it was intended to extend the construction techniques of low-rise housings piled-up till then into the much more storied housings for the sake of high utility of the small land. At first four-storied building was planned. For this purpose testing facility named "Strong Room" was built to do static loading tests of full-scale specimens. Also in the field static and dynamic loading tests were carried out on full-scale four storied specimens.

Joint methods were improved. Dry joint was proposed for the horizontal joints in addition to wet joint.

In 1962 the first four-storied apartmenthouse made of precast concrete was built. After this in 1965 "Recommendations for the Design of Walled Precast Concrete Structures" intended for the four-storied building was published from A.I.J.. In this year a five-storied precast concrete house was realized already and a prestressed four-storied walled precast concrete house was also built. At last in 1971 "Design Guide for the Walled Precast Reinforced Concrete Five-storied Apartmenthouses" was announced by building Center of Japan (B.C.J.). A typical example of five-storied apartmenthouse is shown in Fig.2.

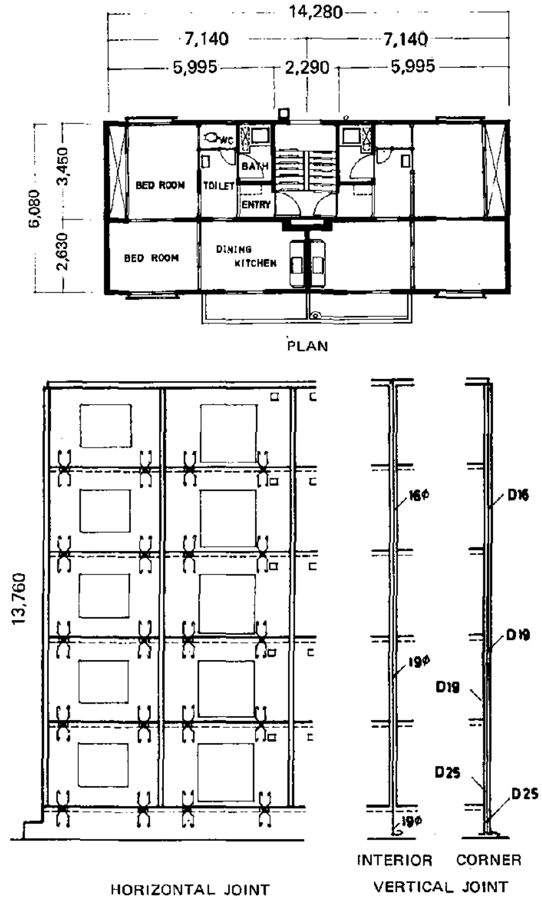

Fig.2 Five Storied Apartment House

Realization of High-rise Housings

Population concentration into big cities in accordance with economic development has caused the upheaval of land cost, so that the housings in this area were obliged to be built much more densely and highly. On the other hand the shortage of laborer in the highly growing economy inevitably strengthened the direction of abbreviation of labor also in the construction industries. With these backgrounds industrialization of construction became important.

Under such circumstances "Competition of the Technological Proposals for Pilot House" was carried out in 1970 and 1971 under the co-auspices of Ministry of Construction (M.C.), Ministry of Industry and International Trade and B.C.J.. Again in 1973 and 1974 "Project for Ashiyahama High-rise Apartmenthouse by Industrialized Techniques" was carried out under the co-auspices of M.C., Hyogo Prefecture, Ashiya City, J.H.C., Hyogo Housing Supply Corporation and B.C.J.. Many new technical proposals were done in these programs.

The representative structural techniques which have enabled the high-rise precast concrete apartmenthouse are H.P.C. technique, walled precast concrete technique, R.P.C. technique and P.S. technique. H.P.C. technique uses H-shaped rolled steel in columns and beams, setting precast concrete walls and slabs into or onto the frame.

Table 1 Joint Methods of Bars

Type	Method	
Welding	Pressure Welding	
	Arc welding	
Mechanical	Bundling	
	Wedging	
	Sleeve	Screw
		Grip
		Explosion
		Filling-up

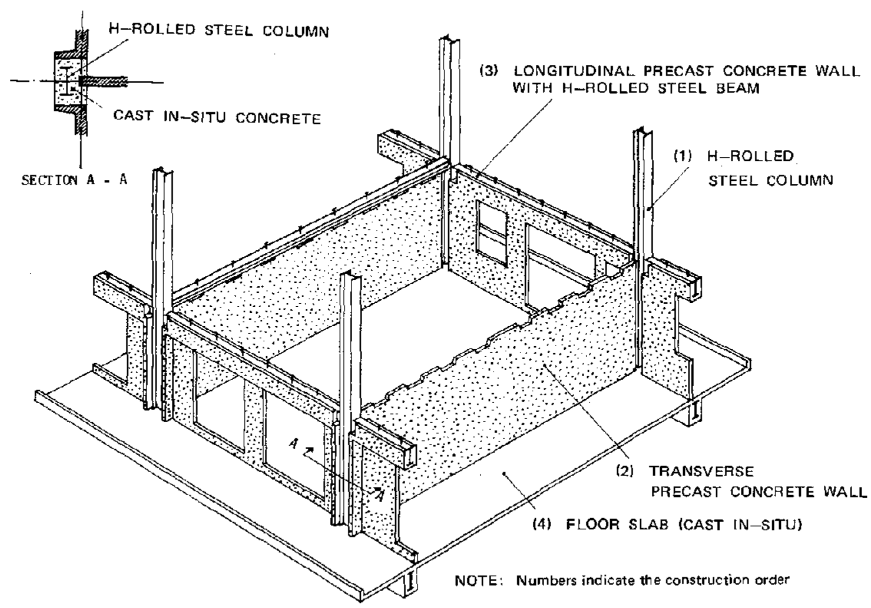

Fig.3 H.P.C. Technique

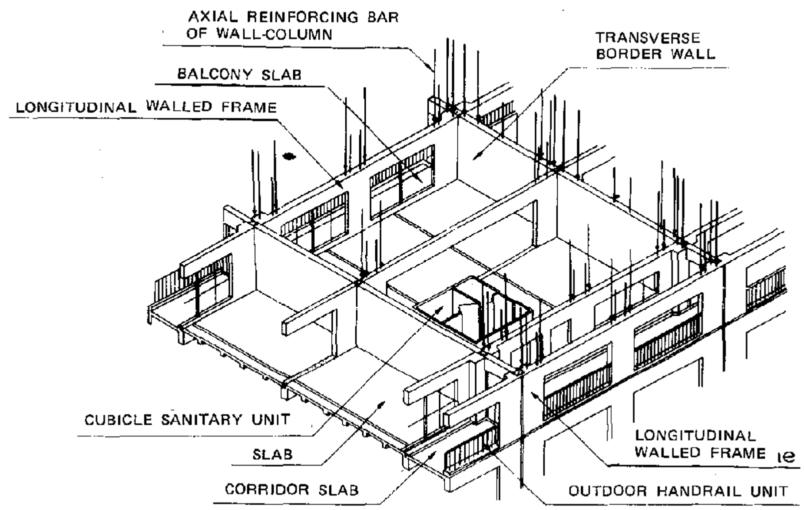

Fig.4 Walled Precast Reinforced Concrete Technique

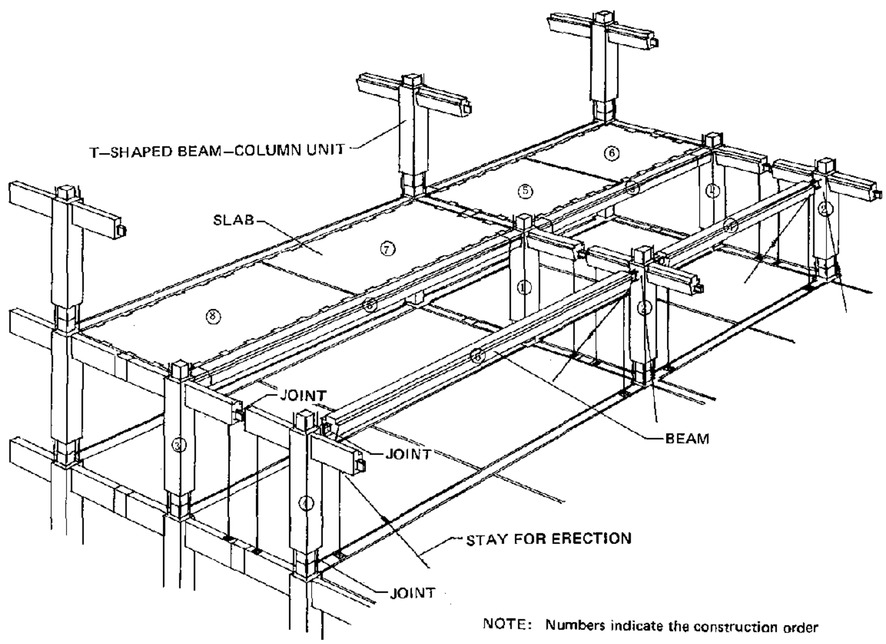

Fig.5 R.P.C. Technique

It is intended for more than 10 storied buildings. Fig.3 shows the schematic view of this method. Walled precast reinforced concrete technique is the method to build 7-15 storied structures by assembling large-sized precast concrete wall panels and slab panels. In general the longitudinal and the transverse directions are composed of walled frames and shear walls respectively. Fig.4 shows its scheme. R.P.C. technique has been projected mainly for 11-14 storied frame-type precast reinforced concrete structures by assembling precast columns, beams and slabs as shown in Fig.5. P.S. technique has been adopted for 7-10 storied walled or framed structures.

The reinforcing bars should be connected firmly to each other in the joints of precast reinforced concrete members in case both of walled and framed structures. The joint methods of bars already developed count nearly 50 in all, largely grouped into welding type and mechanical type as shown in Table 1.

Photo 1 Curtain Wall

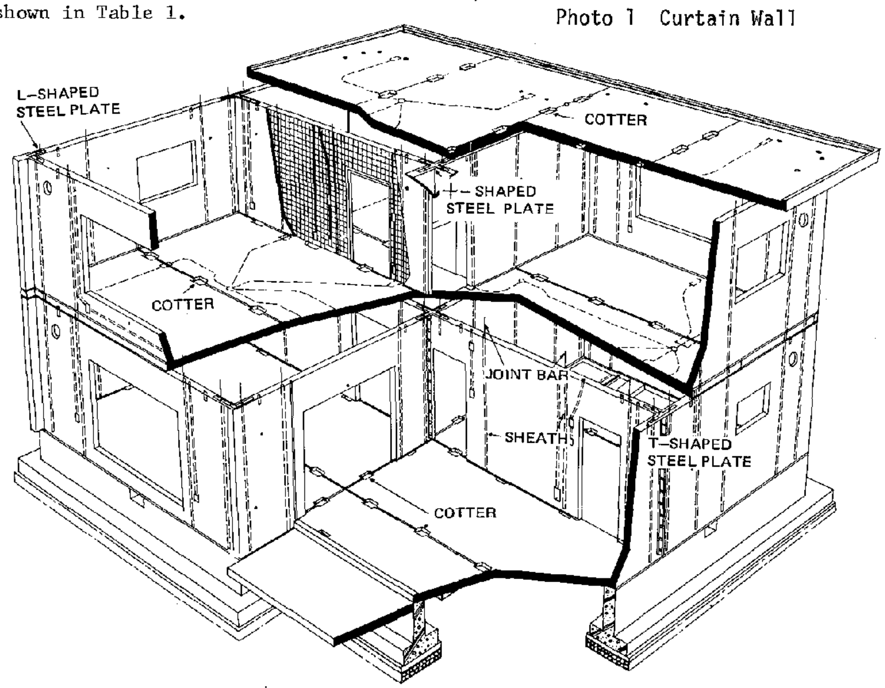

Fig.6 Independent House (PALCON)

Besides the usage of precast concrete members as main structural components mentioned previously, they can be used for non-structural elements such as curtain-walls as shown in Photo 1.

<u>Present Status</u>

Oilshock in 1973 has pushed the declining world economy deep into the depression and strengthened the embarrassing status of stagflation (price upheaval under depression) which has never been experienced. At the same time in Japan the problems of pollution and/or environmental destruction caused by large scale development projects had been socially taken up, and from such a point of view there was growing the atmosphere of reflection against the large scale development.

Till then large scale development of housing lots and construction of gathered housings had been done for the purpose of housing supply in large amount. Precast concrete structures had also been developed and improved chiefly to meet such concentrated needs for housings. However according as such large scale developments disappear in the economic and social conditions described previously, there generated surplus of labor and cost of transportation increased severely. It is unprofitable for precast concrete structures, and conventional cast in-situ method has come to take part instead.

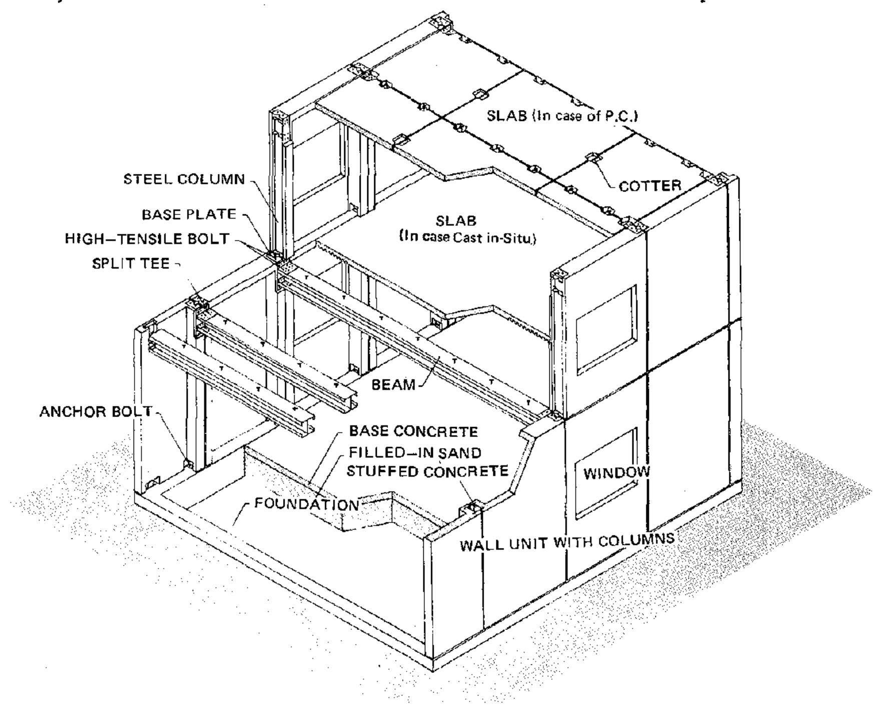

Fig.7 Small or Middle-size Shop (PAL-SHOP)

Though in such a condition, the merits of a precast concrete structure could rather be clarified: reliable quality control, short time of works on site, scarce possibility of environmental trouble by noise and/or dust, and effective utilization of limited sites in densely inhabited areas. The problem is how to attain the cost reduction in production. For this purpose mass-production is indispensable. Up to date the precast technique has been applied mainly to housings including private houses as shown in Fig.6. However recently middle or small size shops, school facilities and government offices have attracted notice, which are large in number and have similar plans and dimensions. There is a movement to sum up such dispersed demands into mass-production. In Fig.7 is shown a skillful example of structure with light-gauged steel and precast concrete panels, which is used for middle or small-size shops.

In 1975 and 1976 "Competition of Technical Proposals for G.S.K. System (School Facilities Construction System)" was carried out aiming at the quality improvement, cost reduction, time saving, development of common members and parts and encouraging the local industries. This system separates a building into eight sub-systems on the idea of open system which makes us possible to build a structure assembling various kinds of parts selected in a market. The sub-systems are structure, exterior wall, roof, partition, ceiling and lighting, interior decoration, electricity and electonics, and machine and sanitation. Again in 1976 and 1977 "Collection of Technical Proposals for Government Office-building Development System" was done on the same idea.

PRECAST CONCRETE SCHOOL BUILDING SYSTEM

Background

In Japan a large number of new schools are in demand in big city areas because of a school population boom. It has been caused by population concentration, a baby boom in the past, and increased ratio of senior highschool-going pupils, which is not a compulsory education in Japan. Most schools are public, and they are generally constructed by a small contractor under a supervision of community officials. But, technology levels of such contractors are varied, and the quality of a school building is not always satisfactory.

In such circumstances, a new school building system, which brings out cost saving, high quality, rapid construction and easy supervision, is urgently required. One of the reply to the need is the G.S.K. system already referred to. And TAISEI CORPORATION has also developed a unique system named Pal-School.

To develop a new school building system, characteristics of Japanese education and school buildings as follow should be considered.

(1) Teaching methods are traditional, and the style of most school buildings are similar: self contained classroom boxes are arranged in a row along a single loaded or double loaded corridor. Then, standaradization of a school building is easy.

(2) Since land cost is very high especially in such areas, most school buildings are 3 or 4 stories high.

(3) Earthquake resistant design is one of the most important factors since Japan is located in the region of highly active seismicity. A moment resisting frame in a longitudinal direction and a frame wall structure in a transverse direction give a typical design style. In Tokachioki Earthquake 1968, columns in logitudinal frames failed in shear, because shear span to depth ratio of those columns was very small (≤ 2.0) on account of wall girders. Then shear failure of a short column came to be a big problem. On the other hand, the frame wall structure in a transverse direction was not damaged, and such type of structure was recognized to be a superior earthquake resistant system.

Outline of Pal-School

Under these background, Pal-School system has been developed. A schematic view of Pal-School is shown in Fig.8.

(1) Both faces in the longitudinal direction consist of precast concrete panels (1, 2, 3 in Fig.8) The width of the panel is 2.25m, and which makes an unit in this direction. This is a quarter of the standard length of a classroom (9m), and decided in consideration of the transportation of elements: Transportation of things more than 2.25m wide

on road is prohibited by the law in Japan. Other than regular class-rooms, two units give a proper length for a staircase, a toilet and so on, and six units compose a specific class room.

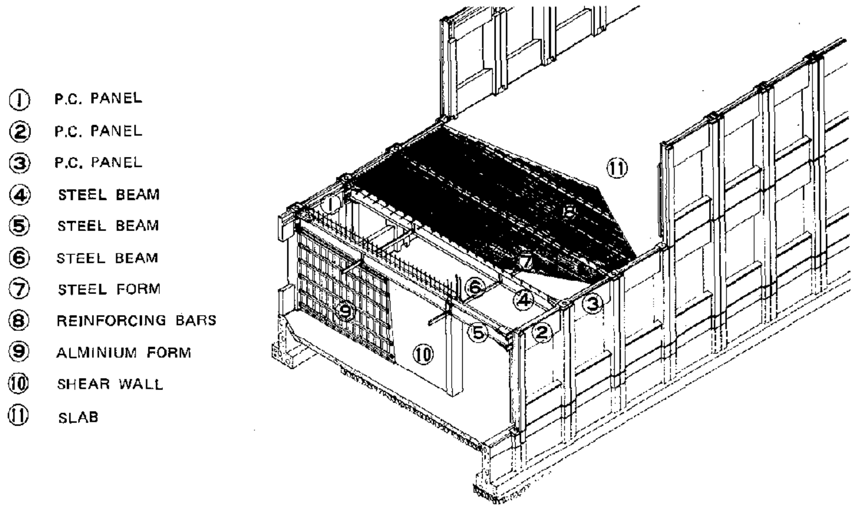

Fig.8 Schematic View of PAL-SCHOOL

(2) An H rolled steel is used for a beam in the transverse direction. (4, 5 in Fig.8)

(3) Slabs (11) and shear walls (10) are reinforced concrete cast on site. Form for the slab is a waved steel plate (thickness is 0.6mm) placed on a light truss which is hung on the steel beams.

The precast concrete panel is 2.25m wide and 3.70m high as shown in Fig. 9, and which is a closed frame with an opening; an alminium sash is fixed in a factory. The colomun is steel reinforced concrete, and the section is 45cm x 30cm. The upper and the lower beams are reinforced concrete both with the section of 18cm x 70cm. Sanded lightweight concrete is used for the precast concrete panels and cast-in-situ floors and shear walls with a compressive strength of 210kg/cm^2.

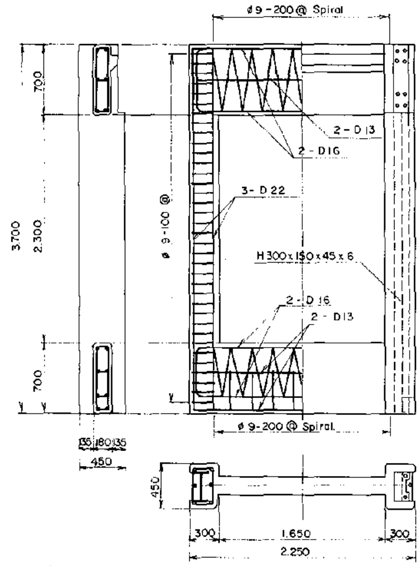

Fig.9 Precast Concrete Panel

Sand and gravel concrete with a compressive strength of $180 kg/cm^2$ is used for footing and 1st foor slabs.

The details of the connection are shown in Fig. 10. Top plates and base plates welded to H rolled steels in the columns are bolted together with a joint plate in between. A steel beam is bolted to the steel in the col umns with split-tees. Concrete is filled in the vacancy between the adjacent columns at the time slab and shear wall concrete is cast. High tensile bolts are used except for the connection between a footing and a panel.

Prodedure of this system is as follows;

(1) The footings and first floor slabs are constructed by the conventional method.

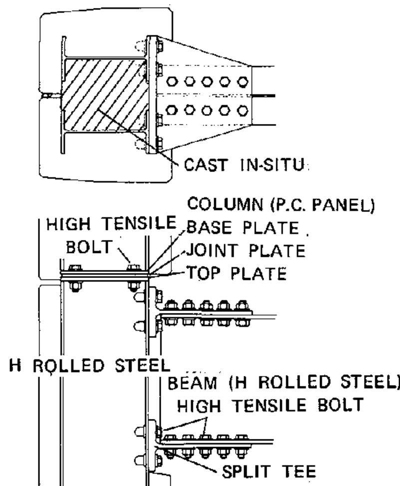

Fig.10 Details of Connection

(2) Precast concrete panels are manufactured in a factory(Photo 2) and transported to the site.

Photo 2

(3) The panels are erected making outer surfaces in the longitudinal direction. (Photo 3)

Photo 3

(4) Steel beams in the transverse direction are set and bolted to the top of column steels. (Photo 4)

(5) Reinforing bars are arranged and forms are set for slab (Photo 5) and shear walls.

(6) Concrete is cast for them (Photo 6) and filled in the vacancy between two adjacent columns of the panels (Fig. 10).

(7) The same works are repeated in every floor.

In this system, joints between elements are simple and reliable so that the assembly is speedy. But it should be noted that the precision of manufacturing and construction is the most important factor. Slab and shear wall are not prefabricated, mainly because cast-in-situ concrete is lower in cost than precast concrete component for those parts now.

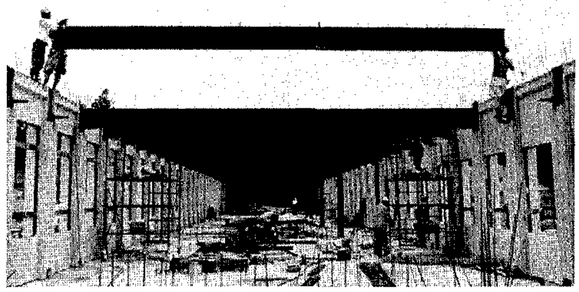

Photo 4

Photo 5

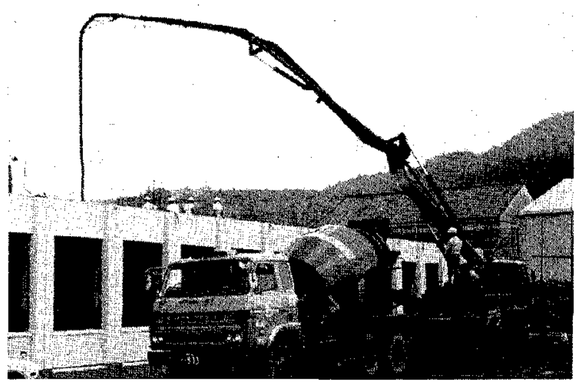

Photo 6

Structural design

Our first planning of this system was a little different from the present one; The column of the panel was an H rolled steel covered by concrete without any reinforcing bars for bending. With the joint system shown before, the two H rolled steels of the adjacent columns were expected to be coupled and to co-operate against bending moment. Then the strucutural system in the longitudinal direction had been estimated to be a moment resisting frame with reinforced concrete beams and coupled H rolled steel columns.

A full-scaled model test was conducted to evaluate this system. The precast concrete panel for the test is shown in Fig. 11, and the loading method is shown in Fig. 12.

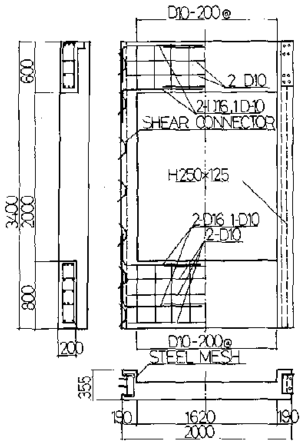

Fig.11 Precast Concrete Panel for Test

The column was an H rolled steel covered with concrete, and steel mesh was arraged around the steel to couple the steel and the concrete. Shear connectors were welded to the web of the column steels to couple the adjacent columns. Four panels were erected on a steel base fixed on the testing floor, and transverse steel beams were bolted to the top of the columns with split tees. The specimen was subjected to lateral load reversals at the top, and the strain of the column steel was measured to evaluate the mechanism of the coupled column.

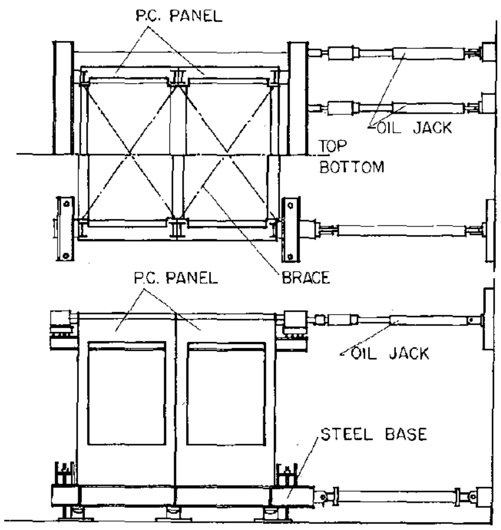

Fig.12 Loading Method

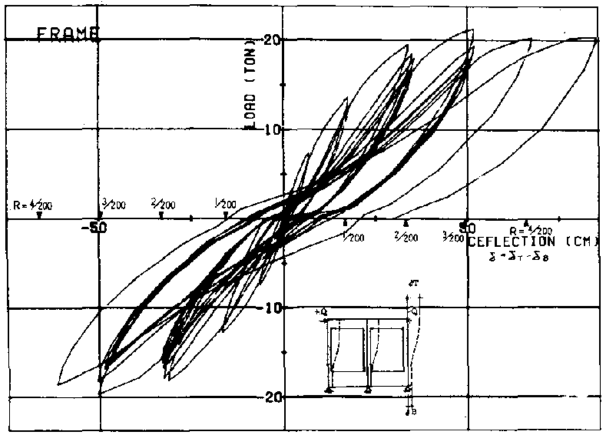

Fig.13 Load-Deflection Curve

The obtained load-deflection curve is shown in Fig. 13. The specimen yielded at column ends, and a very little strength decay was observed under load reversals in a large deflection after yielding as shown in Fig. 6. No symptom of destruction was observed at the joints, and this frame proved to be a ductile moment resisting system. On the other hand, co-operation of the coupled column turned out not to be expected except only at the biginning of the loading.

Then fundamental concept of the structure was changed; Each panels resist the lateral load individually, and the column was designed as a 'self-contained' steel reinforced concrete as shown in Fig. 9.

Structural model for design is shown in Fig. 14. The longitudinal structure consists of pin-jointed frames of the precast concrete panels, and the transverse structure is a frame of a steel reinforced concrete columns which consists of two columns of adjacent panels, and the base of the column is pin-jointed to the top of a column below or to the footing.

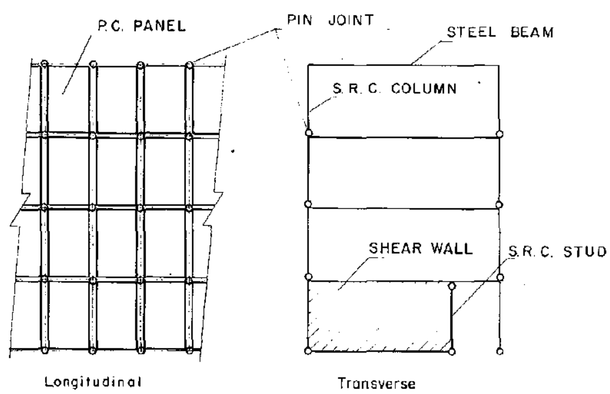

Fig.14 Structural Model for Design

Shear walls are placed in this frame, and on one end of the wall is located an SRC stud.

Design base shear coefficient is 0.3 (for most cases, it is 0.2), and working stress design is applied according to A.I.J. structural standard.

An earthquake resistant characteristics are examined on "The Daraft of the Standard of the Earthquake Load". Since the transverse direction is a frame-wall structure and the safety of such a structure during a severe earthquake have been clarified through the experience of Tokachioki earthquake, only the longitudinal direction was examined. The model structure was Shimada-Gakuen High School referred to later, and a plan of which is shown in Fig. 15. The plan is repetition of same patterns so that an unit pattern (1 panel width) is modelized, and elastic response characteristics were evaluated by modal analysis. Shear force coefficient at an yielding point Cy, that of an elastic response Ce, estimated ductility factor μ_y and story drift index R are shown in Table 1.

The 1st natural period in elastic range is 0.45 sec, and shear force coefficients at yielding are 0.51 - 0.94. In this method, criteria for safety are response ductility factors, story drift indices of each stories and an average drift index. This structure surpassed all these criteria, and proved to be an superior earthquake resisting system.

Table 2.

Story	Cy	Ce	μy	R
4	0.94	1.06	1.13	1/514
3	0.55	0.92	1.89	1/276
2	0.51	0.84	1.83	1/207
1	0.66	0.95	1.53	1/192

$\bar{R}$ = 1/256

Example.. Shimada-Gakuen High School

One of the example of this system applied to a high school building is introduced in the following. Shimada Gakuen High School for 900 pupils consists of north block (3F) and south block (4F), and the total floor area is 4,800m^2 (Fig. 15, Photo 7). The planning is a typical one for a Japanese highschool: self-contained classrooms are arranged along a corridor located on the north side of the building.

Construction time was about 6 months, and two months time saving was attained by the application of this system.

Other than schools, this system can be applied to kindergartens, dormitories, offices, hospitals and so on.

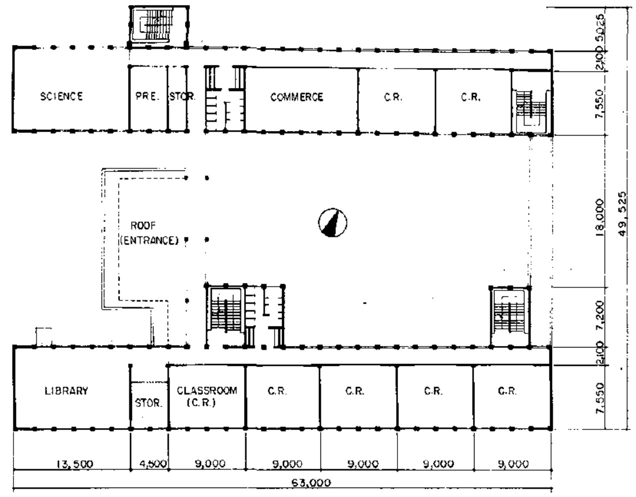

Fig.15 2nd Floor Plan of Shimada-Gakuen High School

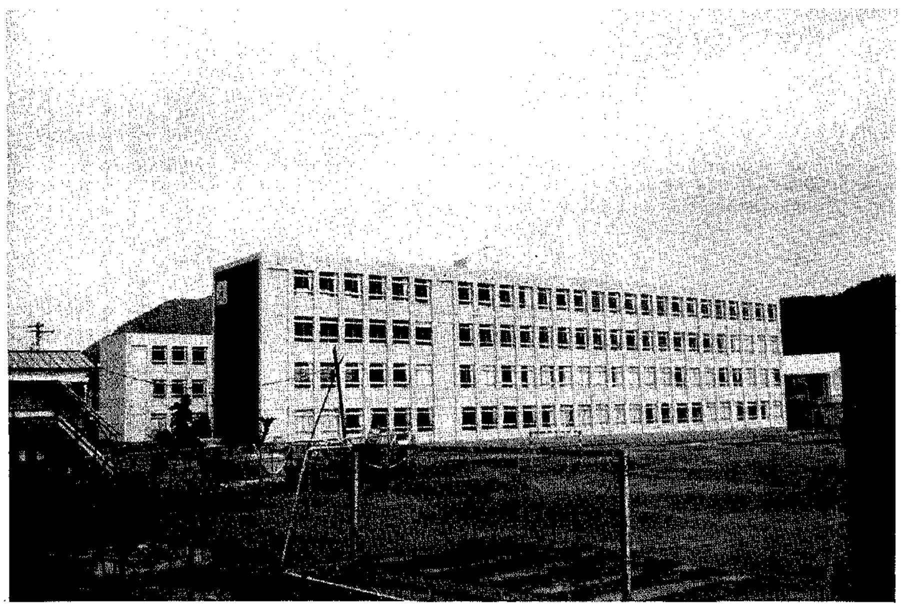

Photo 7 Front View of Shimada Gakuen High-School

CONCLUDING REMARKS

This paper summarized the history of precast concrete structures in Japan, and introduced a new building system developed by the authors as an example of the latest state of the art. Then, characteristic problems still needed to be investigated are discussed in this part as concluding remarks.

General Problems of Precast Concrete Structures

The problems yet to be investigated concerning the structural design of prefabricated reinforced concrete frame structure were referred to in the report of RPCJ*1 committee. The authors will summarize them as general problems of precast concrete structure.

(1) Since precast concrete elements are generally cast in a horizontal position, the bond strength of upper reinforcements may be lower than that of lower ones, because of the settlement of concrete. Though stiff-consistency concrete is cast under highly systematized control so that the settlement may be very little, there are very small amount of data of the bond characteristics of precast concrete elements and such data are indispensable for the design. To get the above data, systematic experimental stucies should be conducted using specimens with variables of casting direction, slump, curing condition and so on.

(2) Strength and deformation characteristics of beam-column connections have not yet been clarified even in traditional reinforced concrete structures, especially on the effect of orthogonal members and slabs. In precast concrete structures, they are much more complicated because of the existence of joints. Systematic experimental studies should be conducted to clarify the characteristics of beam-column connections.

(3) Shear strength required for slabs to transfer lateral forces in an earthquake and details of the connections between precast concrete slabs have not yet been settled. And there are still unknown parts in the effect of slabs on the stiffness and the strength of beams. To clarify those characteristics, experimental studies should be conducted using specimens with typically detailed joints.

(4) Shear transfer characteristics of connections between precast concrete panels subjected to shear only have been clarified through various studies. But in case of connections between beams and columns, they are subjected to combined shear, bending and axial force, and there are still unknown factors especially when the member is subjected to large shear, a small amount of longitudinal reinforcements are arranged, or the shear span to depth ratio is small. Analytical and experimental studies for the subject are required. Moreover, construction standard should be established since construction quality largely affects the shear transfer characteristics in connections.

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*1 RPCJ Committee was organized in Building Center of Japan, and studied "Problems of Structural Design of Prefabricated Reinforced Concrete Frame Structure" from 1973 to 1975.

New Type of Structure

The authors initiated the development of composite structural system as a new type of industrialized building as shown before. In this system, most suitable materials and methods are selected for each parts of the building: steel, concrete or others, industrialized or traditional.

In the case of Shimada-Gakuen High School, materials and methods were selected as follows. The exterior wall pattern was repetitive and concrete was estimated to be most suitable for exterior material for its durability so that precast concrete panels were used. By the application of precast concrete panels, rapid construction and high quality were attained. Moreover cost reduction could be possible to give the panel double functions : a structural element as well as an exterior wall. Connection between elements were rapid and reliable because an H rolled steel was cast in the panel, and which served as a structural element as well. Since the transverse span was long, an H rolled steel was selected for a beam for the purpose of reduction both in depth and weight. Shear walls and slabs are cast on site mainly because the construction cost is lower than the case precast concrete elements are used, and to confirm the structural integrity as well.

Such a composite structural system seems to be one of the suggestions of the future of an industrialization of buildings.

The composite structure contains characteristic problems still needed to be investigated.

(1) Since the composite structure consists of various kinds of materials and structures, it may be erroneous to apply working stress design method because their structural characteristics are different. So, the design method based on elasto-plastic characteristics of the structures should be established.

(2) The structural systems in two principal directions are different, and each joints are detailed for either direction. So, the response characteristics of the structure under two dimensional horizontal motion in an earthquake are complicated and have not been clarified. Analytical and experimental studies should be executed to confirm the enough earthquake proofness.

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METHODS FOR REPAIRING AND RETROFITTING (STRENGTHENING) EXISTING BUILDINGS

by
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INTRODUCTION

The present state of the art for repairing and strengthening existing structures employs methods which have largely been developed through experience and thus are empirical in nature. Because of past limited need for such work, the existence of well established standards or firms which specialize therein and thus maintain the capability to design, develop, test, and apply optimum remedial procedures is limited. Following every major earthquake, however, vast numbers of "over-night experts" seem to appear. Accordingly, due to the infrequent requirement for seismic damage repair in any given area and lack of guide codes or recommended procedures, owners, their engineering consultants, and the controlling authorities are often restricted in utilization of the most optimal methods, and less than desirable results are often obtained.

In the case of seismic damage repair, the exact requirements or objectives of a given program are often quite obvious, i.e., those portions of the structure needing repair have been clearly defined by having failed or received significant damage. In the case of strengthening of existing buildings however, the engineer must depend upon inspection, analysis, and to a very large degree, engineering judgement to determine the areas of weakness that are to receive attention. In either case, existing building codes, in general, do not address themselves toward remedial work, though often requiring any such work to upgrade the particular structure to full code compliance. This frequently results in employment of other than optimal remedial methods and, in some cases, total demolition of the structure rather than appropriate repair or strengthening. Thus the present state of the art is relatively restricted to employment of established methods which are, at least to some degree, covered by existing codes. Such restrictions very much limit the ability of the engineer and constructor in effecting optimal as well as economical restoration.

CURRENT PRACTICE

Present practice generally involves strengthening of existing elements, addition of new force resisting elements, or a combination of the two. In addition, the anchorage of non-structural elements (wall claddings, ornamental components, etc.) is of prime importance.

Strengthening Existing Elements

Existing elements are generally improved by increasing their cohesive nature through injection of grout or other structural adhesive, containing their mass by encasement, increasing their dimension by the addition of section, or a combination of the above. Occasionally they may be braced by the addition of ties, struts, or other connecting elements. Shear walls are often improved by the addition of section. Filling in of existing openings is also a frequent expedient. Strengthening of roof and floor diaphragms usually involves increasing their thickness or the addition of stiffening ribs. Foundation elements are improved by increasing their plan dimensions, extending their depth or both. In addition, underlying soil is sometimes stabilized.

New Elements

In addition to strengthening existing shear walls, new shear walls are frequently constructed. Such new walls often replace existing interior walls which in older buildings are frequently of a non-structural nature. Load transfer to such new walls is generally through existing strengthened, or in some cases, new floor and roof diaphragms. Where required, the addition of new drag members to transfer lateral forces to the shear walls is frequently made.

Design Criteria

Optimal design for strengthening or repair involves a great deal of judgement and, in many cases, engineering decisions which are subjective in nature. [1] However, because of present code requirements, most remedial work employs methods that are at least to some degree established, or are covered in existing building codes or previously approved code exceptions. Additionally, large scale field tests are often utilized in order to obtain code exceptions for specific projects. [1,2,3,4] Due to the time, cost, and effort required to obtain such waivers, the use of new, innovative or unusual procedures is greatly discouraged, however.

REPAIR MATERIALS

Established remedial procedures involve use of several basic materials. Properties of some of these materials such as concrete are well established and generally understood. Others, however, are the result of fairly recent development and, accordingly, much remains to be learned about them.

Shotcrete

Shotcrete, known also as gunite or pneumatically applied concrete has been used for repair as well as new construction for many years. Its properties are fairly well known and provided for in most building codes. Shotcrete may be applied by either a "dry mix" or "wet mix" process. The dry mix process involves premixing of the cement and sand and transfer to the work site through a hose in a stream of compressed air. The end of the hose is equipped with a suitable nozzle at which point water is injected and mixed with the material as it exits at high velocity. The water content can be adjusted at the nozzle and is restricted to approximately that required for proper hydration of the cement. Because shrinkage is virtually eliminated due to the low water content together with the high impact force at which it is applied, properly installed dry mix shotcrete possesses very high bond strength. Compressive strength of 27,600 kN/m^2 (4000 psi) is commonly obtained. The process is particularly suited to restoration work as the material is transported to the work site through hose making its placement virtually unlimited by access restrictions.

Wet mix shotcrete is a somewhat newer development and involves pumping of a premixed cement mortar to the work location. At this point a blast of compressed air diffuses the mortar and impels it on to the substrate surface. In order to move the plastic consistency material through the hose, higher water contents are required, resulting in greater shrinkage potential and diminished bond capability. Additionally, its tendency to sag results in voids on the underside of horizontal reinforcement and obviously diminished bond. Accordingly, the wet mix process is less suited to and has therefore been seldom used for restoration work. Ongoing development of rapid setting additives which will likely overcome the limitations, combined with the greater economy of the wet mix process, could however, result in a considerable increase in its future use. Type "K" (shrinkage compensating cement) has been used in many instances. Beneficial results from such use however are questionable. [3]

Preplaced Aggregate Concrete

Preplaced aggregate concrete is cast by pumping mortar into the pore spaces of previously placed large aggregate. There is virtually no limitation to the maximum size aggregate although the smallest size is usually on the order of 1 cm (3/8 in). The intrusion mortars most commonly used are composed of portland cement, fine sand (Minus No. 8 mesh), and an expansion promoting additive. Epoxy resin materials have also been used though they are generally restricted to work in which very rapid cure times are required. Because each piece of large aggregate in preplaced aggregate concrete is in intimate contact with the adjoining pieces, and the intruded mortar expands after placement, the material is to some degree self stressing and therefore provides high bond strength. It is therefore well suited for restoration type work especially where access is difficult or an unusually great congestion of reinforcing or other inserts exist.

Epoxy Resin

Epoxy resin materials are available in a wide variety of types and consistencies which provide a nearly infinite range of cured physical properties. In general, they offer very high bond strength and for this reason have received wide usage and, in some cases, been acclaimed as a "magical repair material". Actually, epoxies comprise an extremely complex family of chemicals and whereas properly used they are advantageous for a variety of applications, they none the less possess many limitations. The word "epoxy" actually is descriptive of a chemical reaction, specifically the linking of the ethelyne oxide ring with a reactant material. [5] Because of this very broad definition there are literally an infinite number of chemical formulations that can properly be considered epoxies. Accordingly, it is not possible to describe properties that are typical of all potential epoxy formulations. However, most of the commonly used epoxy structural adhesive systems possess properties of thermal expansion and elasticity quite different than concrete. In general, epoxy systems become brittle when cold and soften with an increase of temperature, although such variation differs widely between various systems. Many epoxy materials will not bond to moist or wet surfaces, although they are very satisfactory in a dry environment. In repair of fine cracks a relatively low viscosity material is advantageous, however, in thicker cracks a more viscous material would be desirable in order to prevent leakage and provide better control of its placement. Absorption into the substrate must also be considered when used adjacent to a porous mass. Most common epoxy formulations are subject to creep. Whereas this area is not well understood, it is generally believed that the creep rate increases with an increase in temperature. Because on most of the applications connected with repair or strengthening, only a small portion of the potential strength of such material is generally utilized on a sustained basis, potential creep is probably of limited importance. However, any proposed application which will subject the epoxy material to a continuous stress level more than about 15% of its ultimate strength, should be approached with great caution. Such application would probably merit separate testing of the specific proposed formulation, under actual conditions of usage.

Most epoxies used in strengthening and repair work are two-component systems. They are usually mixed at the time of usage with a pot life varying from about five to thirty minutes. The pot life is dependent upon temperature, being extended as the temperature is lowered. Because the reaction of most epoxies is exothermic, the quantity of material mixed at any one time considerably affects the pot life.

Proportions of the components can vary from 1:1 to 100:1 or greater. Most systems commonly used for repair and strengthening work, however, involve ratios of 1:1, 1½:1, or 2:1. Ratios of 100:5 are not uncommon, however, particularly in formulations which are suitable for use in very cold or otherwise hostile environments. Many established epoxy systems will properly cure at freezing temperatures or below (the freezing temperature of some systems is lower than 0°C (32°F). However, when working in such conditions other factors such as the existence of ice often control.

Commonly used epoxy formulations nearly always have strength characteristics greater than concrete or masonry materials with which they might be used. As with most other properties of epoxies, the strength potentials vary greatly. Typical values for formulations most commonly used in remedial work however are as follows:

	kN/m^2	psi
Compressive	27,600 to 103,500	4,000 to 15,000
Tensile	20,700 to 55,200	3,000 to 8,000
Bond	13,800 to 27,600	2,000 to 4,000

Modulus of elasticity varies greatly between different systems. It also varies within any given system due to variance in temperature. It is none the less significantly lower than that of concrete in most commonly used formulations. At 21°C (70°F) modulus of elasticity values will generally be on the order of 1,380,000 to 2,760,000 kN/m^2 (200,000 to 400,000 psi). The values will raise with a decrease in temperature, and lower as the temperature increases. The magnitude of such variation significantly differs between different formulations.

Exposure to fire or a high source of heat considerably reduces the strength of some epoxy systems. For instance it was determined by Plecnik, et al., [6] that the strength of one particular system tested was reduced essentially to zero at 200°C (398°F). Thus for a concrete wall of 15 cm (6 in) thick containing a crack .25 mm (.01 in) wide, filled with that formulation, a two hour exposure per ASTM E-119 could result in a strength reduction of approximately 80% during exposure. After cooling to ambient temperature, however, this would be reduced to approximately 40%. The effect of fire or heat on different epoxy formulations varies widely and the state of the art does not at this time permit absolute prediction of the effects on different systems.

There are innumerable ways in which an epoxy formulation may be "cheapened" such as by the addition of solvents, diluents, extenders, fillers, and so on. Top quality epoxy resin material systems should contain 100% reactive solids.

As can be clearly seen, when using materials with such an enormous range of physical properties and application requirements, a number of special problems can arise. Whereas it would be impossible in a single paper to enumerate all such potential problems, discussion of some of the more significant and often experienced follows:

Improper Proportioning-- As aforementioned, the ratio of epoxy base resin to hardener varies widely with different formulations. Whereas some systems can tolerate fairly large variations from proper ratio, many others require very precise proportioning if they are to cure and perform properly. When batch mixing is used, weighing of the constituents is recommended, especially with formulations requiring a large differential in the quantity of each component.

When automated equipment is employed only properly calibrated positive displacement pumps should be used. This becomes increasingly important when the mixed resin is to be subjected to differential application pressure such as pressure injection into cracks in concrete. Many problems and failures have resulted from improperly proportioned epoxy materials. [7]

The efficiency of practically any pump will vary according to the pressure head. Because such pressure variations are common in most injection type applications, the use of high efficiency pumps coupled with frequent checks of the material used is mandatory. It is for this reason that the use of "gear" or other non-positive displacement type pumps should be prohibited in injection type work. Specification of epoxy formulations which have proven capable of withstanding substantial variation in proportioning is also advantageous.

Mixing--The various components of any epoxy formulation must be thoroughly mixed if the cured system is to perform properly. Because the basic chemistry requires every particle of hardener to connect and join with its epoxy counterpart, special attention must be directed toward this item. Insufficient mixing of the resin system will result in weak spots where the molecules are not firmly attached to each other, adversely affecting its strength and durability.

Cleanliness of Substrate--The substrate to which epoxy resins are to bond must be free of dirt, grease, laitance, or similar contaminants. A great many failures have been experienced where proper preparation has not been performed expecially where the epoxy resin is being used as a coating. When the substrate is concrete, the preferred method of cleaning is chipping or sandblasting to sound material. A frequently used procedure involves etching with an aqueous acid solution. Where this method is used, it is imperative that all remaining traces of acid are flushed off and entirely removed prior to application of the epoxy. Because the method leaves a moist substrate, the resin system must be compatible therewith. A great many problems have been associated with projects where the above were given insufficient attention.

Excessive exotherm--As aforementioned, the total amount of heat generated by an epoxy resin is dependent upon the resin formulation, quantity or mass of resin involved, and the ambient temperature as well as that of the substrate. It is therefore important to select an epoxy resin formulation that possesses exotherm properties compatible with the factors anticipated on each individual application. Accordingly, where large masses of material are involved, especially if the ambient or substrate temperature is high, relatively low exotherm formulations should be used. Conversely, where very small quantities or thin films of material are involved and the ambient or substrate temperature is cold, a high exotherm (hot) formulation will probably be in order.

The results of many epoxy resin applications have been unsatisfactory due to boiling of the resin caused by excessive heat. When specifying an epoxy material, careful attention must be given to the total heat to be generated, the heat absorption potential of the substrate, and any environmental conditions which will affect the work. Where large masses of resin are used, intermixing of a clean sand or gravel is frequently employed. This not only reduces the total amount of heat producing resin required, but also increases the total heat absorption media. In some cases either the substrate or admix material or both are artificially cooled to increase their heat sink ability. In extreme cases, cooling pipes or tubing, through which compressed air or chilled fluid is circulated during the exotherm period, are run through the epoxy mass.

Moisture--Any epoxy formulation used where moisture is present must be compatible therewith. Additionally, special provision must be made to prevent condensation resulting from exothermic heat being entrapped in the mass. Many field problems involving this factor have resulted. Such have been especially prevelant when cementing bolts or steel dowels into core drilled holes. In this regard, the use of a percussion type drill not requiring water, is a preferred method for preparing holes for such usage.

It is unfortunate that "epoxy" has often been promoted as a sophisticated material that provides a magical cure for most everything. Whereas the various epoxy materials do provide unique solutions to a variety of problems, careful consideration must be made to match the proper formulation to each specific requirement. Epoxies are very similar to protective coatings and unless high quality materials are used they will likely deteriorate with time. The engineer therefore must be extremely cautious in evaluating and specifying such material.

Epoxy Ceramic Foam

The perfecting of viable foam generating epoxy based materials is a recent development, such materials first becoming available in 1972. Because of the apparent advantages in use thereof for seismic repair, and the then pressing need relative to repair of damage caused by the February 9, 1971 San

Fernando earthquake, a full scale field research program was performed. [2] The particular family of foams which proved to be well suited to such application is reported to be of epoxy ceramic derivation and is the subject of United States and foreign patents pending. It is a two component formulation. When properly mixed, foam generation initiates within less than a minute. The magnitude of unrestrained volume increase varies from about 7 to 20 times the original volume. It is unique, however, in that the maximum pressure developed even when the mixed resin is completely restrained, is on the order of only 14 kN/m^2 (2 psi) or less. This factor is significant in that it overcomes the problem of damage to existing elements, resulting from high foaming pressures, experienced with conventional expansive resins. The strengths obtained are a function of the exact formulation used, application procedures, and amount of expansion allowed. They range from somewhat over 690 kN/m^2 (100 psi) for a totally unrestrained specimen to greater than 34,500 kN/m^2 (5000 psi) where expansion of less than about .5 times the original volume occurs. Optimal injection requires high shear mixing and heating which necessitates using sophisticated automated proportioning pump, in-head mixing equipment. Such injection procedure is covered by United States and foreign patents.

One of the unique properties of the material is an apparent variation in density, and subsequent strength within any void or void system. In this regard, the material adjacent to and nearest the peripheal void surfaces appears to obtain higher density and strength. These properties decrease with distance from the peripheal surface with minimum values developing toward the center of the void, the result being a relatively lightweight fill, encased in a much stronger cocoon. The material exhibits extraordinary bond strength to most materials and is stable, even under high temperature. It will not burn or support flame. Following mixing of the two components by the previously discussed dynamic heating process, the resultant resin possesses a high degree of penetrability and will, in fact, penetrate cracks on the order of .25 mm (.01 in). Accordingly, it has proven itself to be an advantageous material especially where bonding wall claddings, existing masonry or other non-structural elements to new or strengthened structural elements is involved.

Wedge Type Anchors

Wedge type anchors are frequently used in strengthening and repair work. They come in a number of different proprietary configurations but in all cases maintain fixity by directing a high stress against the wall of the predrilled holes in which they are placed. Most manufacturers provide data as to the pull out and shear resistance of their specific products, however, such data is nearly always based on static load tests. Although such anchors have proven to be quite reliable in general use, a large number of failures have been noted during earthquakes suggesting a considerably diminished capacity under dynamic conditions. Because the performance of such anchors depends largely on installation procedures, it is not certain whether the noted failures are the result of the anchor itself or improper installation. In one evaluation program [12] a considerable scatter was experienced in pull out results, however, the average failure value under dynamic conditions was about 15% lower than under similar static conditions. Much more needs to be learned relative to the performance of this type of anchor. In the meantime the engineer must be extremely cautious and conservative in its use. Also it must be remembered that although the anchors use may be specified considering only their shear capability, in actuality they will frequently be subject to both shear and tensile forces during an earthquake. Accordingly, a tensile failure, although in itself not adversely affecting stability, could well negate their effectiveness to transfer shear forces ultimately resulting in failure.

SPECIFIC TECHNIQUES

Crack Repair

Perhaps the number one consideration in any remedial treatment is the repair of existing cracks. The use of pressure injected low viscosity epoxy resin has become a fairly standard practice over the last decade or so. In practice, the cracks are first sealed in order to contain the injected resin. The preferred sealing material is a thixotropic epoxy, however, both thermosetting wax and cementious sealing materials have been utilized. Provision for injection is generally provided on a spacing slightly greater than the thickness of the member being repaired. The preferred method involves the use of pre-formed plastic injection ports with appropriate stoppers (normally standard corks). (Fig. 1) Another method commonly used, however, involves the placement of a 6 mm (1/4 in) wide piece of masking tape over the crack at proposed injection locations prior to sealing. Before the sealing material has hardened, the tape is removed leaving that portion of the crack exposed. (Fig. 2).

Figure 1 — Sealing cracks and installing plastic injection ports using thixotropic epoxy resin material.

Figure 2 — Sealing cracks with epoxy utilizing 6 mm (1/4 in) wide masking tape to provide openings for injection.

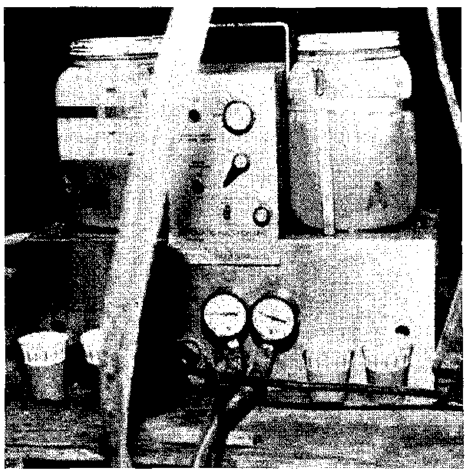


Figure 3 — Proportioning Pump Unit

Figure 4 — In-head mixing and injection apparatus.

Injection is then made utilizing a rubber ring gasket on the injection nozzle which is held tightly against the open crack to prevent leakage. The open crack is then sealed with a paraffin wax material following injection.

Two basic injection methods are commonly practiced. One involves automated proportioning pump in-head mixing equipment, the other batch mixing followed by injection from a pressurized vessel. Although there remains some controversy as to the best method of application, experience has indicated that the in-line mixing system has questionable results when injection of very fine cracks (less than .12 mm [.005 in] is involved, although where applicable it is faster and somewhat more economical. Pressure pots have the disadvantage of tending to hold the exotherm heat with subsequent premature setting of the material. The use of refrigerated pots largely overcomes this limitation however. [2] Because there are wide variations in the properties and proportions of different low viscosity epoxy systems, it is important to match the equipment to the specific formulation when utilizing in-head mixing equipment. Likewise, the properties of the material must be considered and matched to the individual job requirement regardless of the method of injection. A typical automated proportioning pump unit is shown in Figure 3. Figure 4 depicts the in-head mixer used in combination therewith.

Complete and proper injection requires sealing and installation of ports on both sides of the member being injected. Injection is started at the lowest

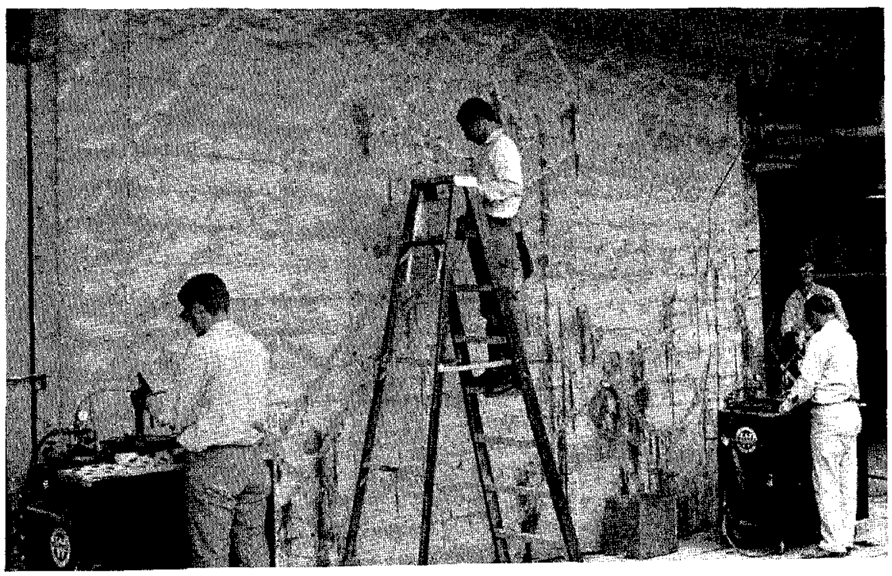


Figure 5 — Epoxy injection utilizing refrigerated pressure pot equipment.

port on one side and continued until resin appears at the next higher port. The injection nozzle is then moved to the next port and the process repeated. Injection ports are sealed as soon as the injection head is removed from them. Likewise the "inspection" or "vent" ports on the opposite side of the member are sealed as the material appears in them. Complete filling of the crack is assured by appearance of the epoxy material at <u>all</u> port locations. The injection phase is, therefore, a two man operation requiring one man on each side of the member. In most instances, a two way telephone system is required to facilitate proper communication. Fig. 5 shows injection in progress on a large wall area. Two crews are at work utilizing refrigerated pressure pot units.

As aforementioned, in order for the epoxy injection to be effective, it is imperative that the cracks be free of dirt, grease, or other contaminants. In relatively new cracks resulting from recent seismic events, satisfactory cleaning can usually be accomplished by vacuuming ahead of the sealing operation. In older cracks special methods including flushing with water or solvents may be required. When flushing materials other than water are used, it is extremely important to confirm their compatibility with the existing concrete as well as the epoxy resin to be used. The use of acids for this purpose has been reported, however, the advisability of such use is questionable, as even with thorough flushing, residual acids may remain. Even minute residues thereof can result in serious corrosion damage to the reinforcing steel. Water blasting has been suggested as a cleaning aid as has blowing the cracks with compressed air. Except in the case of relatively wide cracks [6mm (1/4 in)] and greater, the practice should be discouraged due to the tendency to drive the contaminant farther into the crack. Successful crack repair cannot be made with epoxy resins unless the crack surfaces are clean. Such repair should not be considered for old cracks which are contaminated to a degree that precludes proper cleaning. Where cracks are subject to moisture, the epoxy material used must be compatible with such conditions. Epoxy resins are generally limited to use on cracks with a maximum width of approximately 6 mm (1/4 in). They can be injected in cracks as small as .025 mm (.001 in) or less, however, .10 mm (.004 in) is a more practical lower limit.

Spall Repair

Relatively minor spalls are routinely repaired by shotcrete, epoxy-sand mortar, non-shrink cementious grouts, or standard cement-sand mortar or dry-pack. Where non-shrink grout or cement sand mortars are used, bonding agents of moisture compatible epoxy, polymer emulsion, or neat cement-water paste are sometimes used. It is important that all loose material be removed from such areas and the surface properly roughened and free of contaminants prior to patching.

Shattered Concrete Replacement

Where badly fractured or shattered concrete exists, complete removal and replacement is generally preferred. Reinforcing steel which has been unduly stressed will require correction as hereinafter detailed. Concrete replacement is usually made with shotcrete, preplaced aggregate concrete, or standard portland cement concrete. As previously discussed, Type K (shrinkage compensating) cements are frequently used in such applications.

Filling Non-Visible Voids

Non-visible voids such as rock pockets, honeycomb, or excessive porosity within concrete members, or unfilled joints or cells within masonry infill panels, are frequently filled in strengthening applications. In practice, small diameter holes (approximately 2.5 cm [1 in]) are drilled with sufficient frequency to intercept the voids. The extent and configuration of the voiding can often be established by the injection of compressed air or water into the holes, combined with appropriate monitoring of return locations. In the case of relatively minor voids in concrete, epoxy resin or expansive cement grout has been used. In such instances where the void spaces are small, the cementious mixture generally consists of neat portland cement, water, and an expansive admixture, and is injected in a relatively fluid consistency. Polymer type additives are sometimes incorporated in order to increase bond strength. Such mixtures may also contain very fine sand in a proportion of from 1/2 to 1-1/2 times the cement. Flyash or natural pozzolan is sometimes used to replace up to 50% of the cement.

In the case of larger voids, expansive cement grout or epoxy-ceramic foam is used. Expansive cement grouts used in such instances are similar to those used for minor voids except that they may contain sand up to approximately four times the proportions of cementing material and are generally of a thicker consistency ranging to heavy, mortar-like where large voids are involved. Cement grouts used for such purposes have the advantage of similarity with the substrate materials, and relatively low material cost. Principal disadvantages are the relative high weight and somewhat messier injection requirements. Proper injection of cement grouts requires prewetting of the substrate by injection of water. Accordingly, the excess water must be disposed of and the repaired element will be damp for an extended time period. Such conditions will affect the existing finishes on the element and may render the procedure unsatisfactory in the case of occupied structures. Epoxy-ceramic foams have the advantage of relatively light weight, very high bond strength, and relative ease in controlling placement limits and leakage, due to their generally rapid foaming and set periods. The principal disadvantages are high material costs and relatively low compressive strength. Because of their highly expansive nature (expansion as great as 20 times their original volume) and their high bond strength, such materials have proven extremely useful in the reinforcement and bonding of masonry infill panels especially where the bonding of wall surfacing materials is required.

An extensive research program [2] disclosed that low viscosity resins were generally unsatisfactory for strengthening masonry infill panels due to problems of leakage and absorption. Polyester resins likewise were found unsiutable due to shrinkage. Many common resin foams such as urathane and styrene were likewise unsuitable due to their high expansive pressure and, in some cases, lack of strength.

Bolting, Strapping and Bracing

The continuity between elements is sometimes improved by direct bolting or the placement of steel straps bolted in place across joints or cracks.[8] Parapets, towers, overhanging cornices and similar members are frequently braced by structural steel members which are bolted in place or secured by embedment in replacement mortar concrete or resinous material. [1,8,9]

Where bolting through existing concrete is used, effectiveness can be greatly increased by filling any remaining space between the bolt and hole with epoxy material.

Increasing Section of Existing and Provision of New Elements

Regardless of the particular material or method used for increasing section or provision of new elements, careful consideration must be given to provide for uniform distribution of stress from the new or strengthened elements or assemblies, to the remainder of the existing structure. Special attention should be directed toward tying the floor and roof diaphragms into the lateral force resisting system.

Shear transfer--Provision for shear transfer and bond development must receive adequate consideration and care during construction. In general, all existing concrete surfaces that are to be joined to new concrete should be sandblasted or chipped to a clean, rough condition providing signigicant exposure of the aggregate. In joints which will be subject to high shear, additional roughening with pointed chipping tools to an amplitude of 6 mm (1/4 in) is a frequent requirement. [10,11] In many cases the chipping of keyways may be required. [1] Additional shear resistance can be achieved through the installation of powder driven pins, wedge type anchors and grouted rebar dowels. Where the replacement material is shotcrete or preplaced aggregate concrete, the use of bond coating is not recommended and, in fact, carefully controlled field tests [3]have indicated the use of such actually results in a deleterious effect, when used in combination with shotcrete.

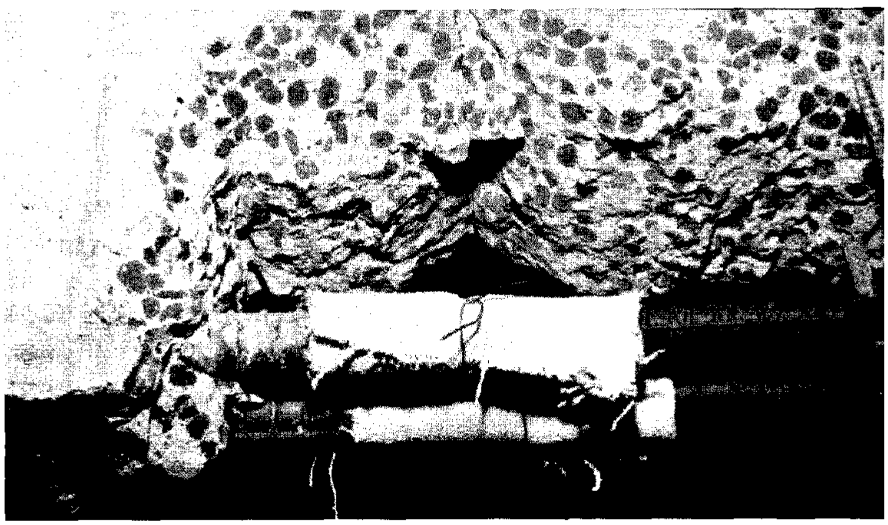


Figure 6 — Rebar weld wrapped in asbestos to prevent rapid cooling.

Reinforcing steel-- Rebar that has been excessively yielded or otherwise damaged, must be replaced. This is generally accomplished by removal of the damaged portions and replacing with new steel welded in place. Generally full penetration butt welding is preferred though lap welding may be used in some cases. In any event, because of the varying heat dissipating properties of the steel which is encased in concrete, and that which remains in the open, such welds will require close control of temperature. Normal procedure involves pre-heating to a temperature of approximately 200°C (400°F) prior to making the weld. Immediately upon completion the weld area should be wrapped in asbestos to prevent rapid cooling. (Fig. 6) Also the concrete should be removed in order to expose the rebar for a minimum of 10 to 15 cm (4 to 6 in) prior to the welding.

In some cases conventional lap joints can be made and in those cases where the reinforcing is in tension only, standard mechanical splices can be used. Where sections to be strengthened are interrupted such as by existing columns or beams, continuity is maintained by either bypassing the steel around the interferring element or continuing the new reinforcing in holes drilled through the existing element.

Rebar dowels--Where it is not possible to penetrate the element such as

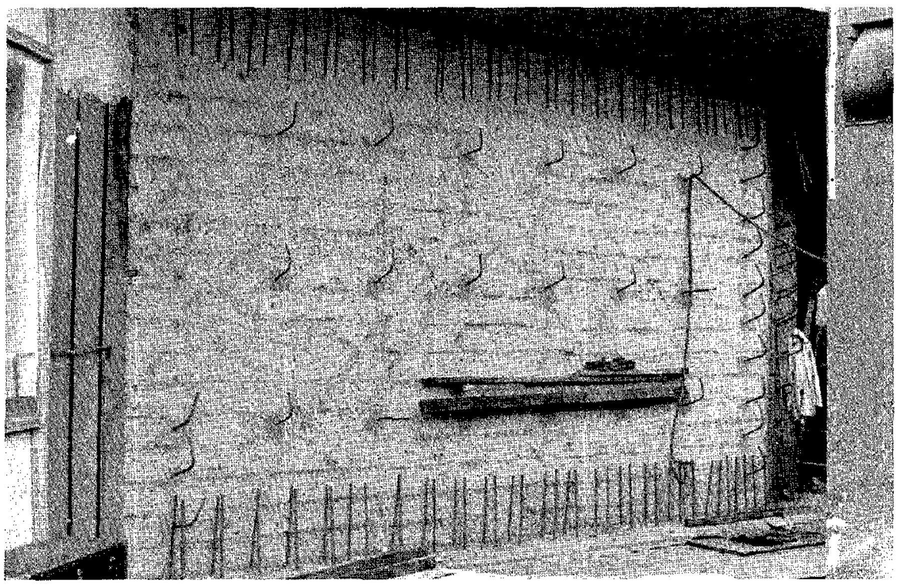


Figure 7 — Rebar dowels in place. Note significant aggregate exposure as a result of sandblasting. (Figure 5 shows same area prior to sand-blast preparation).

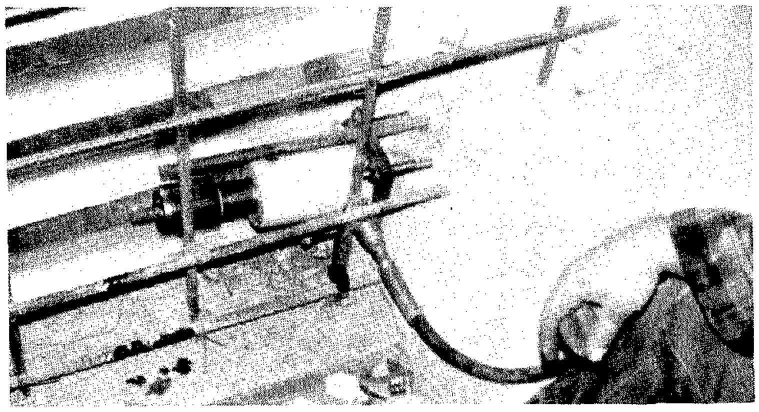


Figure 8 — Pullout test of epoxy set dowel.

in corners or at termini, or where additional shear resistance is required, reinforcing steel dowels are secured in drilled holes. (Fig. 7) Drypack, non-shrink cementious grout and epoxy resin materials have all been used for this purpose. The epoxy resin materials have been proven most suitable [3,9.10] as they require a smaller hole, minimizing possible interference with existing reinforcing, as well as being more economical. Tests have shown that epoxy set dowels properly installed will retain their full yield capacity when embedded approximately ten times their diameter. Because increasing the embedment depth of epoxy set dowels entails only an infinitesimal amount of additional cost, it is practical and probably advisable to so do to at least fifteen bar diameters where thickness of the existing section permits. Field proof testing of grouted bars is frequently required at a rate of from 10% to 50% of the total bars set. (Fig. 8). The frequency of such tests is often reduced however, as the job progresses, if consistently satisfactory results are obtained. Proper performance requires that the holes be filled, preferably from the closed end outward, the bar then being pushed into the partially filled hole so that the resin material oozes out around it, insuring complete contact. The bar is usually twisted slightly as it is inserted in order to accomplish this result. The resin material can be injected with proportioning pump in-head mixing equipment or by hand caulking guns. In either case, the nozzle must be provided with a hose or tube of sufficient length to reach the bottom of the hole being filled. The installation of dowels in horizontal or overhead locations is facilitated by covering the hole with masking tape. A slit is then made in the tape through which the resin injection tube is inserted, followed by the bar, the tape acting as a barrier to prevent the material from running out. Somewhat thixotropic resin formulations are generally used for this work. Optimal hole size is the smallest that can be readily drilled and yet enable insertion of the steel. Because of the creep potential of many epoxy formulations, hole sizes more than about 13 mm (1/2 in) greater than the bar diameter should not be used.

SPECIFIC ELEMENTS

Foundations

Structural repair and strengthening frequently entails improvement and sometimes augmentation of the existing foundation system. Both increased dead load which nearly always results from strengthening operations, as well as potential loads resulting from high overturning forces generated in the new or strengthened shear walls during an earthquake, must be considered. Where the foundation system consists of conventional spread footings or mats, the most frequent treatment involves increasing the dimension, depth, or both of the existing elements. Additionally, new foundation elements are sometimes provided. This is almost always the case when new shear walls are constructed. Fig. 9 shows typical examples of foundation augmentation. Where existing depth is increased, the work usually is done in alternate segments of between 1.5 m (5 ft) and 3 m (10 ft) in length. Conventional concrete or shotcrete is generally used in such work. Continuity is maintained by placing new reinforcing steel through the existing elements, the use of epoxy grouted dowels, or a combination of the two.

In the case of pile foundations, additional piles may be installed or the surrounding and/or underlying soil strengthened. Because of access problems usually involved in such work, additional piles frequently are composed of a number of short sections of steel piling which are welded together. They are jacked into place using the building as a reaction, alternately jacking and welding in additional pieces. The actual pile material may be steel H section or steel tubing. Where steel tubing is used, dirt forced into the interior thereof is sometimes cleaned out and replaced with concrete. Cast in drilled hole concrete piling can also be provided in some cases.

Where strengthening of the soil itself is to be performed, "compaction grouting" [13,14,15] in the case of fine grained soils, or chemical solidification [16,17] in the case of relatively permeable granular material, can be used. Compaction grouting results in densification of the affected soil and has been used to thereby reduce the potential for liquefaction in such soils. Reduction of liquefaction potential in granular soil by providing cohesion through chemical solidification has also been performed.

Strengthening Existing Shear Walls

Existing shear walls are frequently strengthened by the addition of section, most often utilizing shotcrete. As indicated on Figures 7 and 10, integrity of the strengthened mass is obtained by proper preparation of adjoining surface, continuation of the new reinforcing steel through the slabs, epoxy set dowels at termini, and provision of new shear dowels at regular spacings throughout the field of the wall. Fig. 11 shows the completed rebar installation and shotcrete application in progress. Similarly, continuity is maintained at the abutments with existing walls or beams by proper preparation of the adjoining surfaces, installation of epoxy set dowels or continuation of the reinforcing through the abutting element. Where reinforcing is continued through elements, the annular space between the rebar and hole should be filled with epoxy.

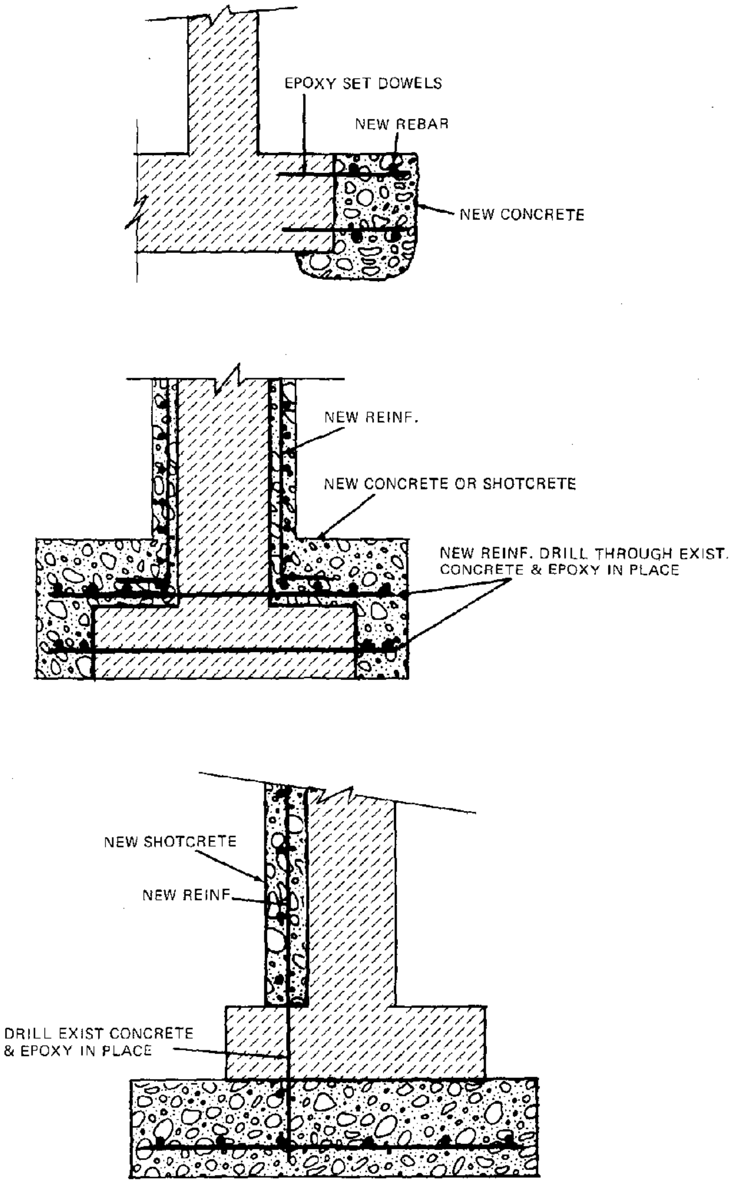


Figure 9 — Typical foundation augmentation.

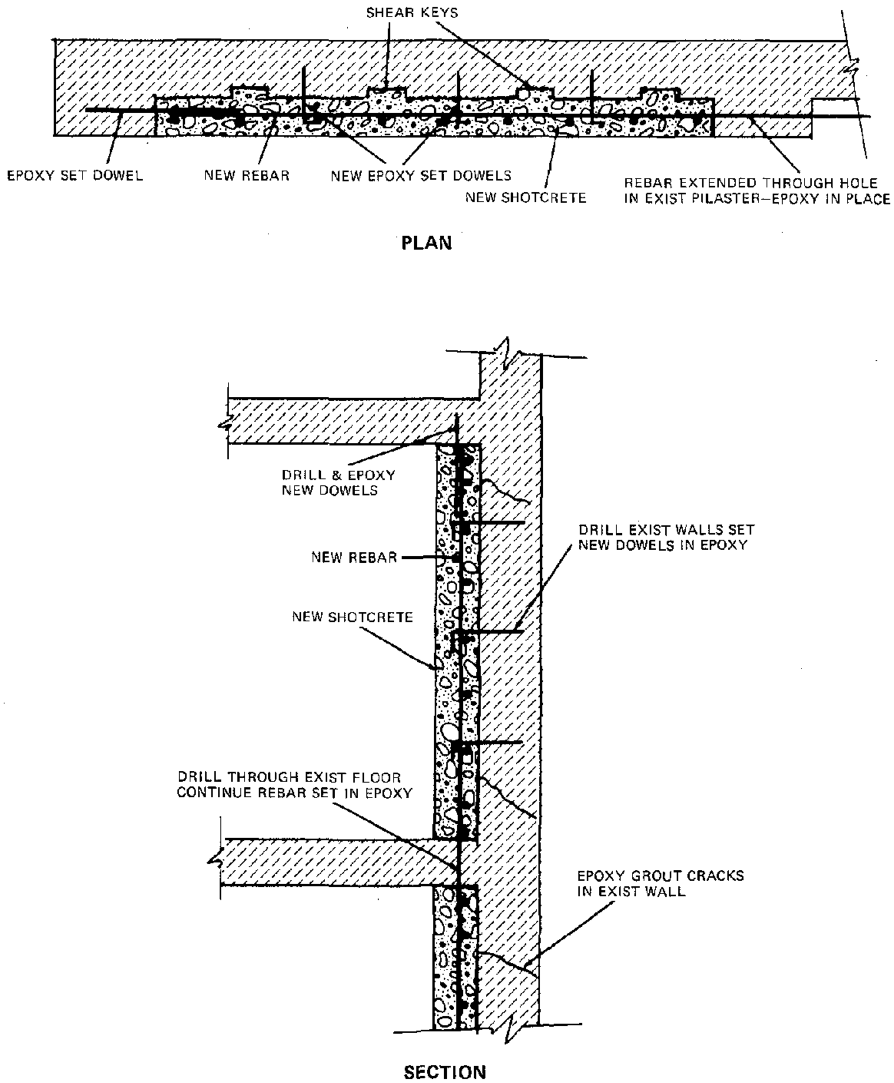


Figure 10 — Typical shear wall strengthening.

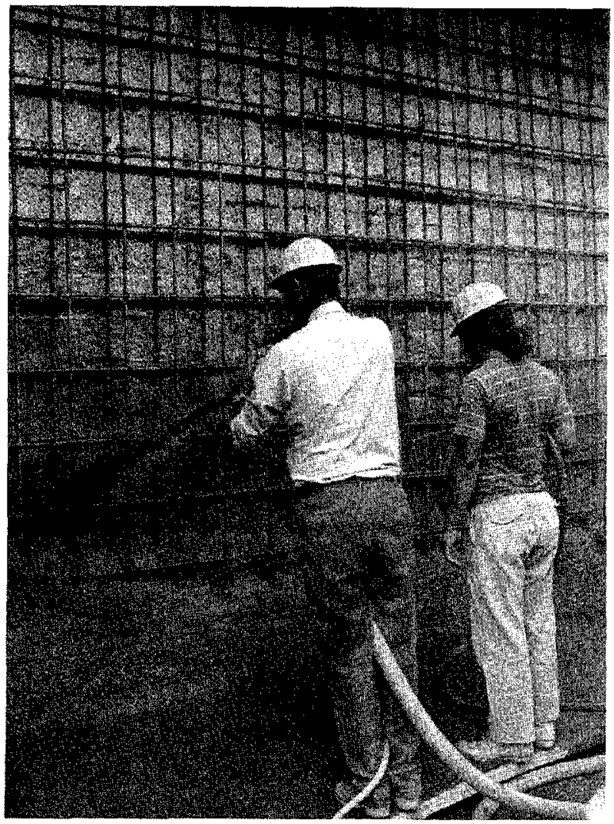


Figure 11 — Shotcrete application. Note complete reinforcing steel and epoxy set dowel installation and well prepared roughened surface with significant aggregate exposure.

A frequent expedient involves filling existing openings in shear walls. This often requires rerouting of mechanical ducts, lines, and other components which frequently penetrate such walls. Where openings are filled in, epoxy grouted dowels usually are installed throughout the periphery.

In the case of concrete frame buildings with masonry infill walls, it is fairly common practice to remove one or two wythes of brick, replacing them with properly reinforced gunite. When this is done thickened "ribs" are frequently provided around openings and at other areas where additional strength is desired. (Fig. 12) By such removal of portions of the existing masonry, it

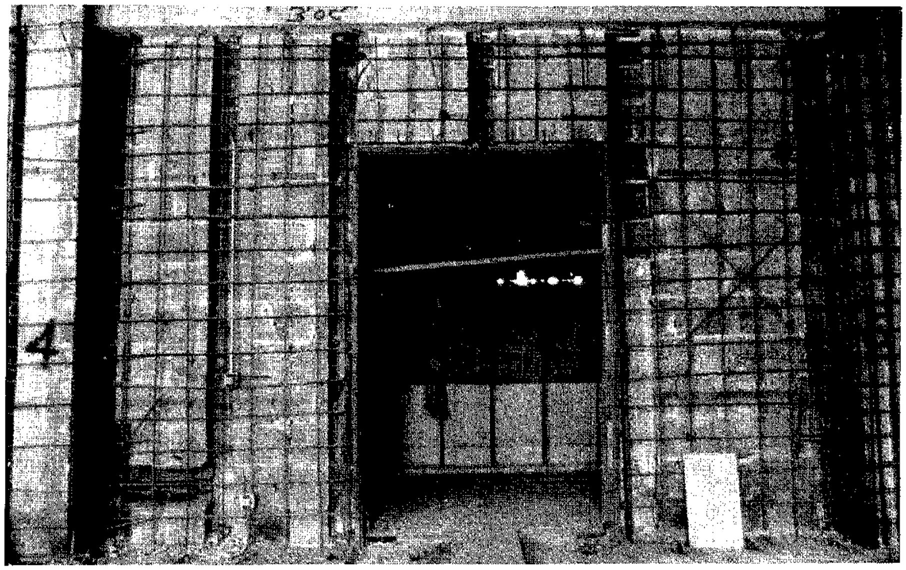


Figure 12 — Masonry infill wall prepared for shotcrete.

is often possible to maintain the original dimension. This also reduces additional weight imposed on the foundation system. In such operations proper anchorage of the remaining wall components must be considered.

Because exterior facades usually are the most decorative and therefore important to preserve, such work is frequently done from the interior of the structure. Accordingly, provision must be made for proper anchorage of decorative elements. Fig. 13 indicates some previously utilized methods to tie ceramic, cast stone, or similar ornamentation to the strengthened structural wall section. As shown such anchorage can be provided by the installation of bolts, wedge type anchors, epoxy grouted bars, and in some cases, injection of epoxy ceramic foam. [1,4,9] Where the exterior cladding is composed of brick, stone, terra cotta, or similar material, provision must be made to prevent its dislodgement during a seismic event. Epoxy ceramic foam injection, as shown in Fig. 14, has proven to be a valid method for such anchorage. [2,4,9] However, expansive cement grouts have also been used. [1.2]

Experimental work has been reported [19] wherein various precast infill panels were installed for strengthening. Wide scale usage of such systems probably is not likely, however, due to the advanced state of development and greater economy of the other established systems. Additionally, provision of new infill panels in themselves would not provide anchorage of existing non-reinforced masonry or decorative wall cladding which, by necessity would require either removal or some type of attachment.

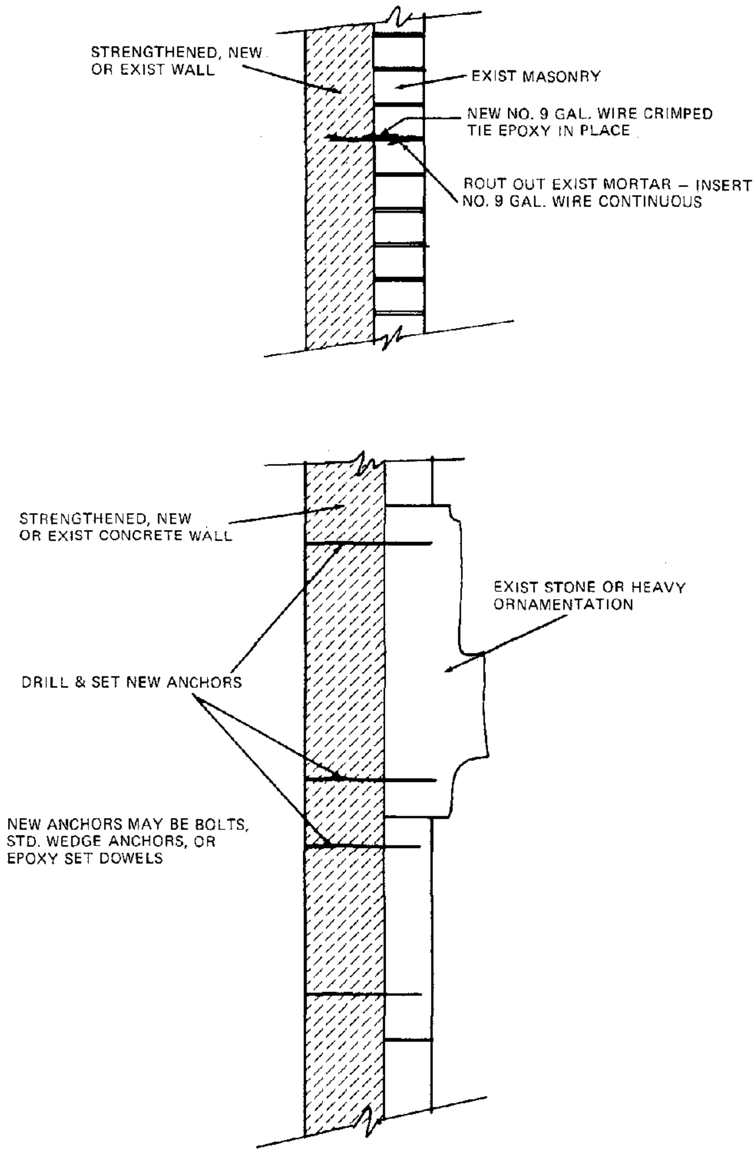


Figure 13 — Typical methods for anchoring exterior claddings.

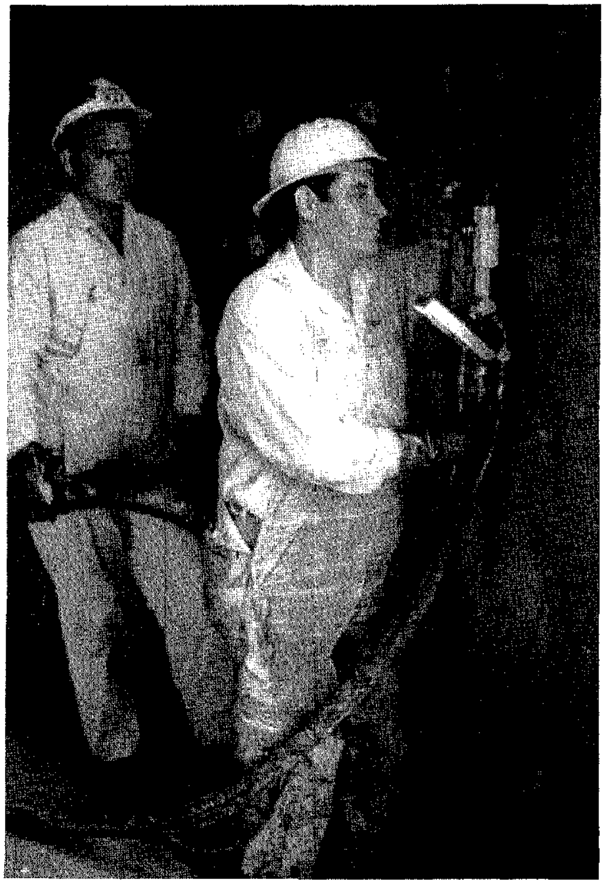


Figure 14 — Injection of epoxy ceramic structural foam through new reinforced shotcrete on interior, in order to secure exterior covering.

Considerable improvement in the strength of unreinforced masonry infill walls has been reported by Wyllie & Dean [18]. Therein they report masonry infill walls were repaired by the addition of wire mesh and plaster, following the 1966 earthquake in Lima, Peru. The building, the Colegio Villa Maria school, was revisited following the October, 1974 event with no damage to the previously repaired building being noted, although other buildings on the site were damaged. It is noteworthy, that whereas such minimal treatment obviously falls far short of current United States code requirements, it does none the less substantially increase the earthquake resistance of the affected elements.

New Shear Walls

New shear walls are generally constructed of conventional reinforced concrete or shotcrete although any material system which will provide the required resistance can be used. Where such elements are cast between existing concrete framing members, continuity of the reinforcing steel or the use of epoxy set dowels can be used in a manner similar to that used for strengthening existing walls. The same methods for preparing abutting surfaces are similarly utilized.

Framing Members

Existing columns and beams are frequently upgraded by the addition of properly reinforced shotcrete. In order to provide a collector system to drag lateral forces to the shear walls, existing beams frequently receive special attention. Additionally, new drag members are often provided. As with the previously discussed work, proper preparation of the surfaces to receive new shotcrete is imperative. New reinforcing steel is placed with special emphasis to insure continuity through or around other conflicting elements. Shear transfer and continuity are provided by the use of chipped shear keys, wedge anchors, or grouted bars. [1,3,4,8,9,10,11] Typical examples of such strengthening are shown in Figs. 15 and 16.

Floor and Roof Diaphragms

Floor and roof diaphragms provide a major contribution to the distribution of forces throughout any structure. Accordingly, in strengthening applications they very frequently will require special attention. Strengthening of existing diaphragms is often accomplished by the addition of an overlay of either concrete or shotcrete. Where "change" in the elevation of the top surface cannot be tolerated, which is frequently the case, the addition of shotcrete on the underside is a frequent expedient. In some cases stiffening ribs can be utilized. Occasionally, new diaphragms can be added by filling in abandoned shafts, stairwells, etc. The removal of existing concrete and total replacement is occasionally made as well. The preparation of surfaces and installation of reinforcing and shear resisting devices is similar to that used in the strengthening of other elements as previously discussed.

Realignment of Displaced Members

Displaced or collapsed members, assemblies or sub-assemblies can often be realigned by structural jacking. [15,20] Unitized jacking equipment is available which permits the use of a nearly unlimited number of individual jacks operated individually or in unison from a central control console. Such equipment provides the ability to precisely realign misplaced elements without the introduction of new or deleterious stresses. Following realignment, the damaged or missing sections are replaced as previously discussed under "Shattered Concrete Replacement". As an example, the beam shown in Fig. 6, as well as those adjacent to it, were jacked to proper alignment prior to rebar replacement.

Ancorage of Non-Structural Elements

Fixity of parapets, cornices, sculptered figures and similar non-structural

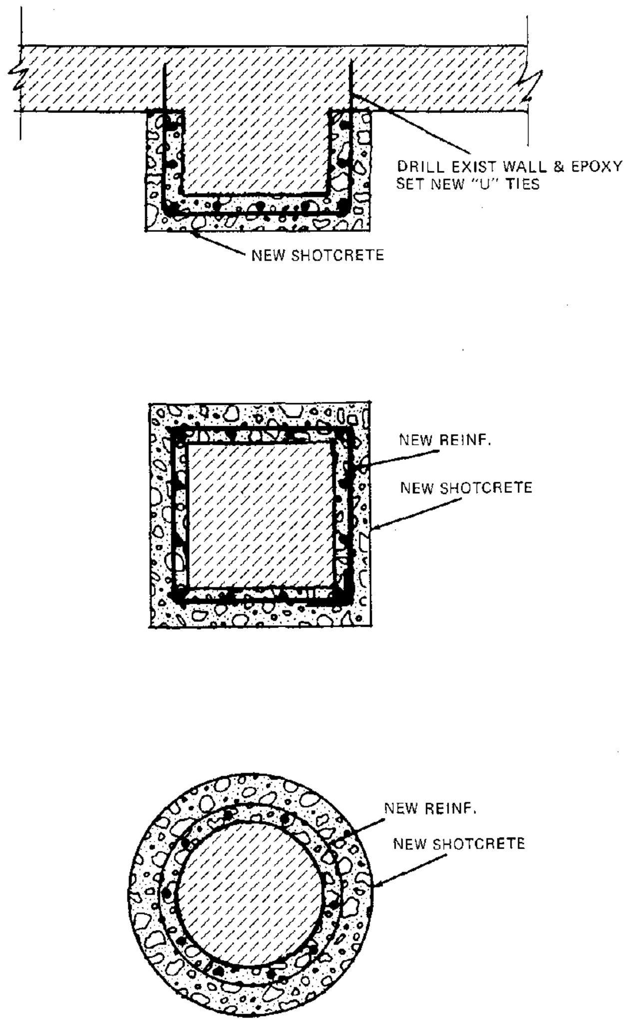


Figure 15 — Typical methods for column strengthening.

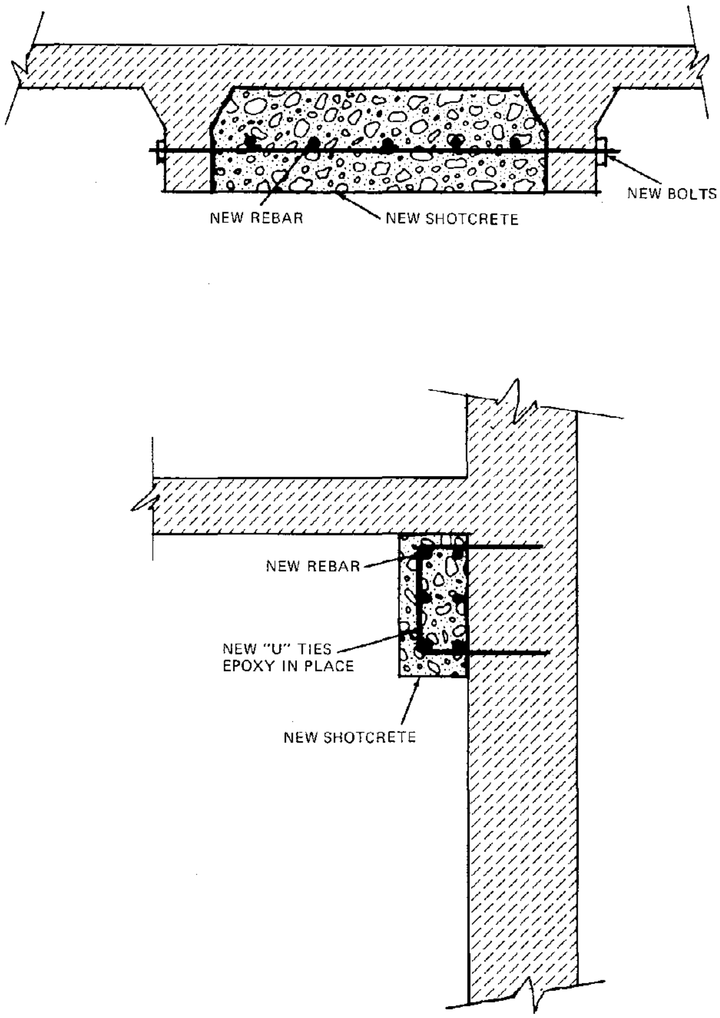


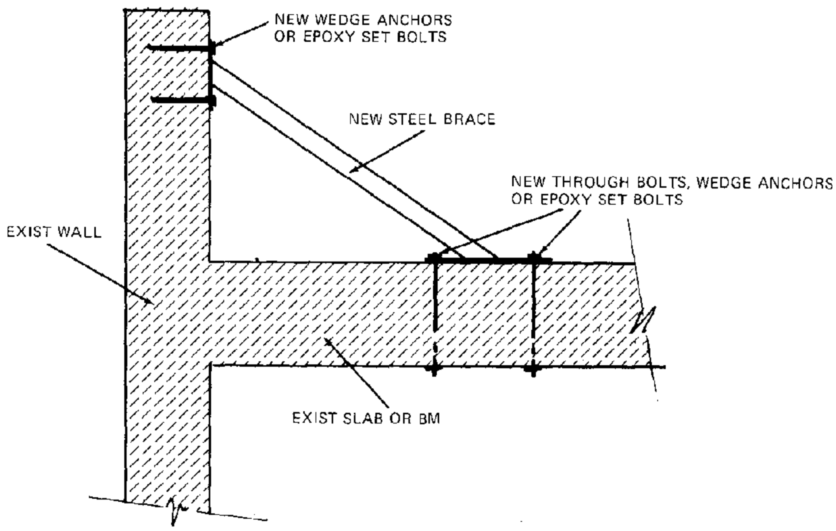
Figure 16 — Typical methods for strengthening beams and new collector members.

elements is required to render a building sesimically safe. Such anchorage may be accomplished by tying with wedge type or grouted anchors, bonding with epoxy mortar or similar materials, bolting, or bracing with steel elements [1, 4,9] Fig. 17 shows a steel bracket cornice hanger during installation. Fig. 18 depicts typical anchorage methods. When steel is embedded within the structure, it is important to assure against eventual corrosion. Hot dip galvanizing is frequently used in this regard. Additional protection is sometimes provided by encasement with concrete or epoxy ceramic foam. [9]

Figure 17 — Steel cornice anchors. Following setting of remaining terra cotta cornice elements, entire section will be filled with epoxy-ceramic foam for additional stability and protection from corrosion.

OBSERVATIONS AND CONCLUSIONS

Whereas historically, neither repair nor strengthening of structures has been performed on a large scale basis except immediately following damaging earthquakes, viable methods and procedures therefor have, none the less, become fairly well established. In most instances, the methods have developed empirically, although in some instances laboratory or field research has preceeded actual usage. Because of existing building code requirements, most of the work performed in the United States has been severely limited as to methodology. Accordingly, the materials and procedures which have become fairly well accepted, if not already covered by existing building codes, have been developed under conditions of considerable restraint, in order to obtain approved exceptions to the controlling code. In the case of large or important projects, often elaborate and costly testing programs have been performed. However, many smaller and less important undertakings and, in some



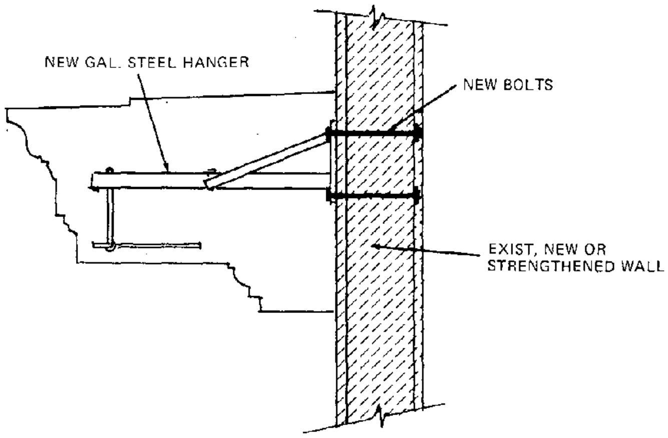


Figure 18 — Typical anchorage for parapets and cornices.

cases, even large projects have been performed using methods that all too often have been inadequate, improper, and certainly not in the best interests of the owners or the public.

Due to the infrequency of seismic events in any given area, when they do occur, design engineers, building officials, and established constructors are severely limited in providing remedial treatment, due to lack of experience in performing such work on a wide scale. Accordingly, formulation of a set of guide procedures for such performance is badly needed.

In the case of strengthening existing structures, methods available both from the standpoint of design and construction, are severely limited by the frequent requirement that any such work conform to current code provisions. Due to such rigid requirements, and the resulting inability of owners to legally make partial improvements, many structures tend to remain in essentially their original hazardous condition. There is, therefore, a need to establish reasonable requirements for the strengthening of existing structures. Such requirements must relate the required level of improvement to the age, occupancy, and general usage of the particular structure. In many instances, a fairly great improvement in the gross stability of a structure can be made at a relatively small cost, even though that structure might continue to fall far short of code compliance. Accordingly, modified code provisions should be formulated which encourage owners to strengthen existing structures within reason. Simplified procedures for acceptance and use of materials, methods, or procedures not covered in present codes should be provided.

Most code requirements and established test procedures for evaluation of materials and structures are based on static methods. To realistically evaluate seismic performance of individual materials, elements, or assemblies, especially where composite construction is involved, appropriate dynamic evaluation is required. Accordingly, the development of improved procedures, standards, and pertinent equipment for large scale dynamic testing is needed.

Present methods for evaluation of existing structures for dynamic stability is largely limited to analysis of the individual components thereof and their computed or assumed interaction. Better methods for determining the behavior of structures and interaction between the various elements, subelements, and individual materials is needed. The effectiveness of many remedial materials and procedures has been fairly well established through their use over an extended period of time and resulting "testing" by actual seismic events. However, many of the relatively new materials and systems have not had this "advantage". There is a definite need to confirm the effective performance of unproven remedial methods, preferably utilizing full scale models under simulated seismic conditions.

Epoxies and other resinous materials have received widespread acceptance in repair work in recent years. However, this field of chemistry is extremely complex and very little is understood relative to the properties or resulting behavior of such materials. Compilation of a guide, enabling identification of specific properties required for desired end results, and development of appropriate analytical and acceptance criteria thereof is needed. Additionally, evaluation of the composite behavior of epoxy injected elements and assemblies, and in particular, their performance under conditions of elevated temperature (fire) or extreme exposure is needed.

Whereas the use of "dry mix" shotcrete has been well established and its value proven, because the "wet mix" process promises even greater flexibility as well as economy, its further development will prove advantageous. Development of special procedures to correct shortcomings therewith, particularly the tendency to sag, needs special attention.

History has proven the capability of engineering and construction professionals to design and perform remedial work in an optimal manner. Building code restrictions have, however, frequently limited the performance of such professionals. Perhaps the number one need to improve the state of the art is to create an optimal integration of these factors.

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METHODS AND COSTS OF REINFORCING VETERANS ADMINISTRATION EXISTING BUILDINGS

by

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INTRODUCTION

The San Fernando Earthquake of February 9. 1971, destroyed or severely damaged four major hospitals. Two patient-occupied buildings of the Veterans Administration Hospital collapsed, killing 46 persons. The Veterans Administration buildings that collapsed were designed and constructed prior to the development of seismic design codes.

In order to prevent a recurrence of such a disaster, a program was immediately undertaken to evaluate other VA hospitals and determine their seismic resistance. Details of the program have been described elsewhere (3), (12).

Consultants were retained to study the geologic and seismic hazards at each site. Where the risk seemed high, other consultants evaluated the ability of the hospital buildings to withstand earthquake forces and suggest methods of strengthening with corresponding cost estimates for those buildings found to be deficient.

From the reports, the VA determines an appropriate course of action at each hospital. In many instances, it may be more appropriate to abandon obsolete buildings than to reinforce them. All decisions are made by the Administrator of Veterans Affairs, based on advice of the Chief Medical Director and the Assistant Administrator for Construction.

This paper summarized the findings of the program to date. A few cautionary notes on costs are offered. The initial studies were begun in 1971 and a few are still underway. Costs presented are those estimated at the time of completion of the individual reports, and need to be adjusted to allow for cost escalation since then. Furthermore, costs reported include both structural and nonstructural costs, although predominately structural. This will be discussed later in this paper. Finally, some of the analyses include costs required to preserve historic buildings.

CRITERIA FOR EVALUATION OF VA EXISTING BUILDINGS

Although the Veterans Administration had followed local building codes until the San Fernando earthquake, the poor performance of the newer hospitals during the earthquake raised doubts as to the adequacy of current local codes for hospital design. Therefore the Veterans Administration appointed a Committee of consultants to develop requirements for earthquake-resistant design of VA hospital facilities. The Committee Members were Bruce A. Bolt, Professor of Seismology, University of California, Berkeley; Roy G. Johnston, Consulting Structural Engineer, Los Angeles, California; Mete A. Sozen, Professor of Civil Engineering, University of Illinois; and the author.

Sixty-eight VA Hospitals are in Zones 2 and 3 on the Seismic Risk map in the 1973 Uniform Building Code. At each site, consultants evaluated geologic and seismic hazards and estimated A_{max}, the peak horizontal ground acceleration for the site during the life of the structures (approximately 100 years).

Building Population

Two hundred and eighty buildings at twenty-six hospital sites were reviewed in this report. The buildings ranged in size from about 1000 square feet (92.9 m^2) to over 600,000 square feet (5574 m^2) and were constructed over a period of time from the turn of the century to 1972. Figure 1 is a histogram of the year of construction of the buildings in this report.

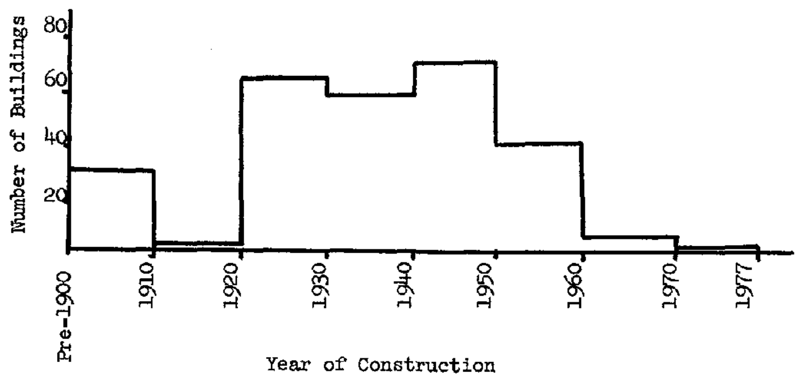


FIGURE 1 - AGE OF BUILDING POPULATION

Two buildings were originally designed for a nominal earthquake force (circa 1950 UBC requirements). None of the others was originally designed to be earthquake resistant. Almost all buildings have unreinforced masonry.

The list does not include pre-1933 VA buildings in California. The most serious problems encountered by the VA were in California. It was evident after receiving reports on the condition of the buildings and the cost of required reinforcement that the agency had no choice but to take positive action at certain Veterans Administration hospitals in California. In simple terms, a large percentage of buildings at these West Coast hospitals were constructed before 1935, when no seismic design requirements were in general use. The buildings were recognized as being unable to withstand earthquakes of high or even moderate intensity. In the face of such circumstances the VA acted immediately to vacate the weaker buildings and to start the improvement of those which had to be continued in service.

For ease of comparison, buildings in this report were assigned to one of five basic structural systems similar to those used by Larrabee (7): (1) Masonry Bearing Wall, (2) Concrete Frame with Masonry Infill Wall, (3) Steel Frame with Masonry Infill Wall, (4) Wood Floor with Masonry Bearing Wall, and (5) Other Structural Systems.

Twenty-one of the buildings are over four stories in height, with the tallest being 15 stories high. Thirty-four buildings have an occupied area greater than 80,000 square feet (7432 m^2).

Fifty-four of the buildings are on sites where A_{max} is estimated as greater than 0.15 g, that is, sites where serious, major ground motion is expected.

And finally, 118 of the buildings are sleeping buildings, and therefore occupied by patients at all times. Most of the rest of the buildings are administration, support, recreation, and other such buildings that are most heavily occupied during the normal daylight working hours.

Analysis

Forces on a building are estimated using the methods specified in "Earthquake-Resistant Design Requirements for Veterans Administration Hospital Facilities" (12 The base shear is:

$$Vo = \alpha \, (DAF) \, A_{max} \, W \qquad (1)$$

in which α = an estimated energy dissipation factor corresponding to the type of construction; DAF is obtained from a design spectrum; and W = total dead weight of the structure. Ambient vibration measurements of VA buildings by the Geological Survey indicate that the period of an existing buildings may be conservatively estimated as

$$T = 0.05N \qquad (2)$$

where N = number of stories.

The distribution of earthquake forces to each story is in accordance with conventional practice. Within a story, the forces are distributed to the resisting elements in proportion to their relative stiffnesses.

The forces are assumed to act nonconcurrently in the direction of each of the major axes of the building.

Calculation of base shear is relatively simple. The difficulty arises when one seeks a reasonable method of analysis of existing buildings. Larrabee (7) studied costs of reinforcing existing buildings and concluded that there are real differences of opinion on strengthening methods and that differences in methodology must be considered as one of the major sources of variance in the cost estimates. Freeman (6) discussed the methodology he adopted in analyzing a major VA hospital in a moderate zone.

It was noted earlier that almost all of the buildings contain unreinforced masonry. And a significant proportion of VA buildings were constructed with reinforced concrete frames and unreinforced filler walls. These buildings were designed for vertical loads only, but may possess considerable lateral strength. Blume (2) cited tests to destruction of two frames of a three-story hospital in South Africa. The frames were not designed to resist lateral forces. The frames were of similar construction, except that one was braced with poor, "loose fitting" masonry infill walls. The frame with the infill walls functioned as a single structural unit, and carried a load much greater than the sum of the capacities of the wall and frame. It failed at a tension splice (designed as a compression splice) in the first-story column.

Unreinforced masonry may have a marked effect on the response of the structure to seismic motion, and may even precipitate collapse of a building. Under some circumstances, the masonry, even though unreinforced, may provide enough lateral resistance for a building to resist earthquake forces safely. Therefore, the masonry's response to both in-plane and out-of-plane forces should be considered.

Engineers have expressed concern that masonry construction, even in a single building, is so variable as to be unreliable. A National Bureau of Standards report for the Veterans Administration (4) presented a methodology for testing and evaluating the strength of unreinforced masonry in a building. Generally speaking, the NBS noted that shear strength was the critical parameter, and recommended diagonal compression tests to estimate shear strength. Further tests by a private testing laboratory, Testing Engineers, Incorporated, demonstrated that tests of cores were also acceptable and costs considerably less than diagonal compression tests (10). However, unless great care is exercised, many of the cores are broken during the coring process.

No generally accepted methodology has been developed for analyzing this type of building. The author proposed a method (8) based on tests of frame wall systems reported by Fiorato (5). In this approach, two failure patterns are considered: (1) those relating to uncracked infill walls, and (2) those that develop after cracking of the infill walls. The latter are modeled by using the braced frame mechanism shown in Figure 2. It is assumed that the cracked masonry forms wedges that brace the upper and lower portion of columns. Plastic hinges are assumed to develop at the extreme points of the wedges. Based on available test results, it is assumed that this model leads to a conservative estimate of the ultimate strength of the frame-wall system.

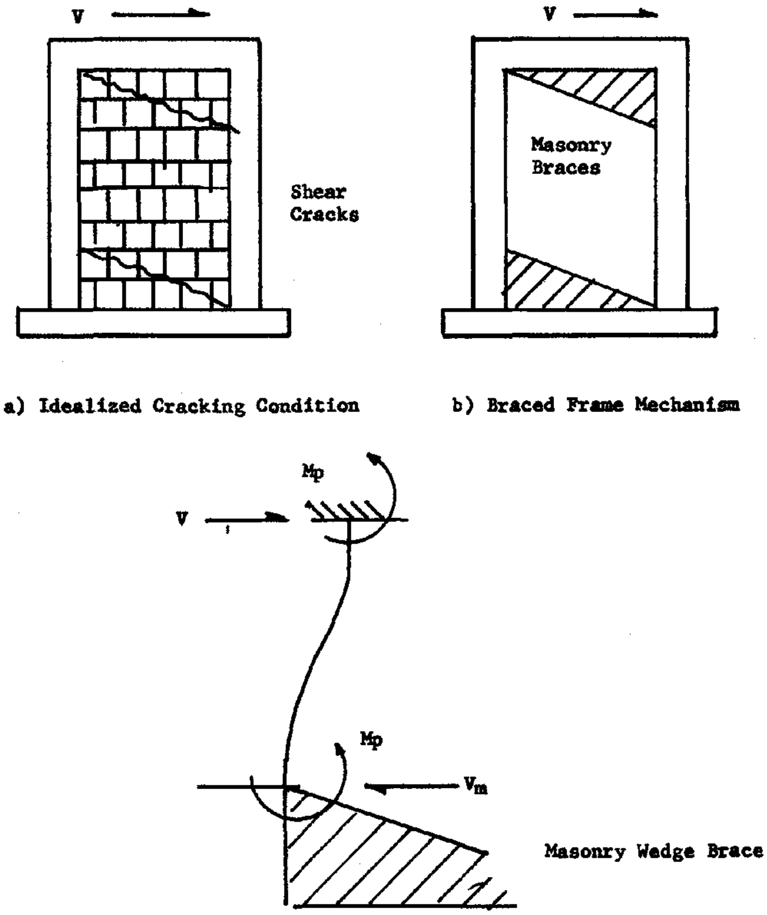
Out-of-plane forces were analyzed by assuming arching action, based on tests by Wilton and Gabrielson (13).

This methodology was used to analyze some of the VA buildings, and the results were somewhat encouraging.

METHODS OF REINFORCING EXISTING BUILDINGS

Scope of Work

The building evaluations in the Veterans Administration program were carried



a) Idealized Cracking Condition

b) Braced Frame Mechanism

c) Idealized Column Mechanisms

FIGURE 2 - SINGLE BAY BRACED FRAME MECHANISM

out in two phases. In Phase I, consultants were asked to evaluate the buildings and identify significant structural and nonstructural seismic deficiencies, classify the building as to whether or not it conforms to the seismic standards, and assign "non-conforming" structures to a category in terms of potential seriousness of the deficiencies. Field inspections and limited test samples of concrete and masonry were taken during this phase.

For those structures found seriously deficient, the consultant was asked to perform Phase II studies. These studies were to develop diagrammatic sketches and cost estimates for correcting the seismic deficiencies noted in Phase I. An important consideration in the cost estimate was to allow for disruption and restoration of all non-structural components to permit the installation of the structural bracing where necessary. Costs of anchoring and stabilizing critical non-structural components was also included in this phase. Complete scopes of work for Phase I and Phase II studies are in the VA Earthquake-Resistant Design Requirements (12).

No restrictions were placed on solutions for reinforcing the buildings except that disruption of hospital operations were to be kept to an absolute minimum. All consultants were encouraged to consider the strength of the existing masonry where appropriate and to consider the level of ground shaking estimated for the site.

Solutions

In a number of buildings, the major deficiency was that floor and roof systems, serving as diaphragms, were not securely anchored to the principal lateral force resisting systems. Corrections in most instances, were relatively inexpensive. At some buildings, wood diaphragms were judged inadequate and replacement was recommended.

The vast majority of buildings were judged to need strengthening of the lateral force resisting system, and new shear walls were recommended. They would be either predominantly exterior or interior walls, or some combination of exterior and interior walls.

A few consultants proposed imaginative solutions, often involving steel bracing systems, but these were limited to special buildings and are not considered generally applicable.

From a review of these solutions, some general guidelines have been developed.

General Guidelines

A necessary first step in reinforcing an existing building to resist earthquake forces is to conduct a careful inspection of the building to identify the principal structural elements, determine the state of repair of the building, and assess the quality of construction.

The construction drawings can provide time-saving information on the sizes and locations of columns, exterior wall construction, length of dowels, and so on. This information should be verified by spot checks in the field. Furthermore, additions and alterations should be noted that may affect the earthquake response of the structure.

The inspection should also identify locations where gravity loads have caused distress. These include settlement of foundations, cracks in structural members, and noticeable deflections of beams and girders. The causes of these conditions should be corrected when reinforcing the building. Often there are shrinkage cracks in a concrete floor or roof slab, and a determination must be made as to whether these cracks impair its capacity to act as a diaphragm.

Finally, the quality of construction is established by sampling and laboratory testing.

Buildings located in areas where moderate to strong earthquakes are likely to occur should have: (1) a lateral force resisting system that is capable of resisting the earthquake forces estimated for the site, (2) diaphragms that are capable of distributing the earthquake forces to the lateral force resisting elements, (3) masonry walls braced as necessary to prevent collapse under strong motion, and (4) other potential hazards (such as loose parapets) corrected.

The amount of corrective work varies from building to building exposure. Furthermore, consideration must be given to the total cost of structural alterations, including the cost of disruptions to operations of the facility.

Lateral Force Resisting System

If the lateral force seismic resistance is inadequate, it can be increased in two basic ways: (1) by introducing masonry interior partitions in order to improve the earthquake resistance of the existing column network, and (2) by introducing new shear walls of reinforced concrete or reinforced masonry construction.

These new stiffening elements should be located so as to reduce torsional problems. Preferably, the elements should be placed symmetrically about the center of mass. Furthermore, openings in a new wall should be kept to a minimum and arranged so as to maximize the rigidity of the wall.

In a number of instances, existing masonry partitions have been replaced by steel or wood stud partitions. This may seriously reduce the earthquake-resistan of the structure. If analysis indicates that the unreinforced masonry is inadequate, it should be braced or replaced. However, new partitions should have at least the same rigidity as existing ones. If necessary, the new partitions should be reinforced, and details should be provided to securely anchor the new partitions to the existing structural columns, beams, and girders.

Introducing new structural partitions may disrupt an area in many ways. Ceilings, lights, ducts and floor areas may be disturbed and operation of the facility interrupted. The cost of the disruption and restoration of these areas must be considered as well as the construction cost of the structural elements. For example, it may be preferable to place new partitions where there is minimal interruption to the building environment, rather than at the locations indicated by optimal structural solutions, provided that the new elements are effective structurally.

The new partitions may be constructed of masonry, reinforced concrete, or pneumatically applied concrete. Masonry partitions are often the least disruptiv but, unless adequately reinforced, may be the weakest. Pneumatically applied concrete partitions require erection of dust proof barriers and there may be a housekeeping problem with the rebound.

New shear walls can be very effective in increasing the lateral force resisting capacity of an existing building. In addition, they can reduce the lateral displacement of the building due to earthquake forces, thus minimizing non-structural damage. If shear walls carry the major portion of the lateral forces, the VA Requirements permit an α factor of 1/3, instead of the 1/2 factor for buildings where infill walls are the stiffening elements.

Interior shear walls may have the same disruptive effects as new partitions on building operations and, therefore, their locations must be chosen with this in mind. Sometimes they can be conveniently constructed in stair towers, although provisions are necessary for the safe egress of the occupants during construction. If the shear wall is constructed between existing columns, as an infill wall, the columns and foundations must be able to resist the axial forces induced by the overturning forces on the shear walls.

Generally, introducing new exterior shear walls is less disruptive to a building operation than new interior walls. The walls can be of reinforced concrete, reinforced masonry, or pneumatically applied concrete. See Figure 3.

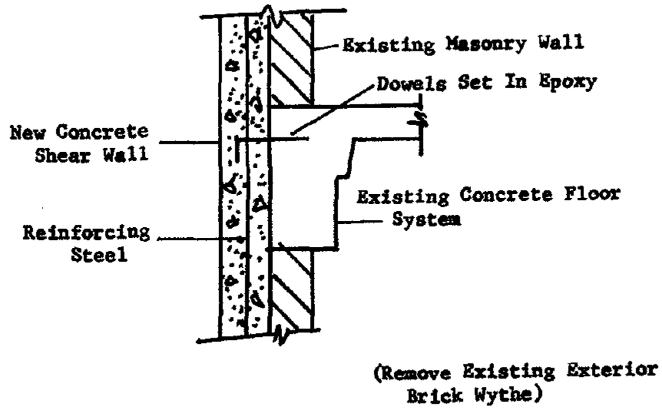
Rather than replacing the entire wall, one method of constructing new exterior shear walls would replace the existing exterior brick wythe by a reinforced grout space and new exterior wythe as shown in Figure 3. The wall then acts as a reinforced masonry wall. The new wall is directly anchored to the frame and to the inner wythe of the existing wall. The existing inner wythe should be cleared, sometimes even sandblasted, to assure good bond between the new and old walls.

The exterior brick wythe is removed by chipping hammers. Interior rooms should be vacated while the chipping is underway since the operation is quite noisy. Window openings can be shielded by sheets of plywood, secured from the inside, and taped around the perimeter to prevent dust infiltration into the room.

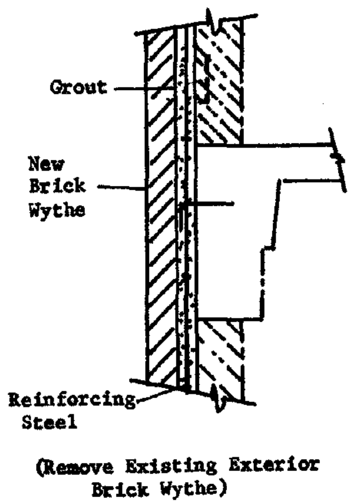
Figure 3c shows another scheme, adding the reinforced grout space and wythe directly to the existing exterior wythe. The analysis involves several additional considerations: (1) the additional weight of the new wall adds to the lateral forces; (2) the center of the new wall is laterally offset several inches from the structural frame and existing wall it supports; and (3) the additional weight and overturning forces may be eccentric loads on the existing foundations. Again, it is important that the existing wall be cleaned and moistened to assure good bond with the new construction. In addition, there should be shear keys or mechanical ties between the walls so that shearing stresses can be transferred from the old wall to the new one.

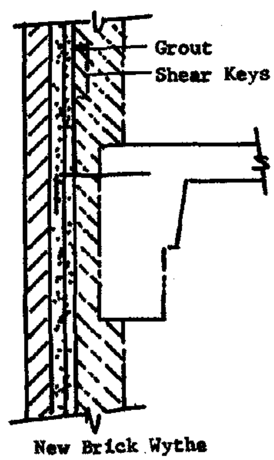
This scheme does not greatly disturb the existing operations; and, the total cost, including foundations, may be equal to the other schemes.

Another possibility that may be considered is to introduce exterior concrete buttresses to brace the building. These may be constructed as exterior stair towers, thus increasing the number of emergency exits from the building. Exterior buttresses require special considerations: (1) massive "flagpole" foundations must be provided to resist overturning forces since the dead load of the building is not acting to resist these forces; (2) the transfer of shear from the diaphragm to the buttress is difficult, and (3) vertical ground motion may create a tendency for the buttress and the wall to respond independently, and impair their functioning together in resisting earthquake motion.



a) New Concrete Shear Wall





b) New Reinforced Masonry Shear Wall

c) New Reinforced Masonry Shear Wall

FIGURE 3 - NEW EXTERIOR SHEAR WALLS

NONSTRUCTURAL CONSIDERATIONS

Until recently, engineers as a rule were not concerned with properly anchoring architectural, mechanical and electrical components nor with stabilizing equipment, furniture and supplies in an earthquake environment. Since the San Fernando earthquake, these important areas have begun to receive attention, but vital "how to" information is still difficult to find. In our opinion, the best guidelines were developed by Ayres (1).

The VA can offer several direct guidelines:

(1) New VA Hospitals in highly seismic areas are designed to continue to function for four days without outside assistance after experiencing a major earthquake. VA sets explicit requirements for emergency electrical power, water supply, sewage disposal and steam supply.

(2) Architectural, mechanical, and electrical components are anchored. In addition, VA design requirements mandate strict draft limits which minimize earthquake damage to these components.

(3) A detailed study on VA hospital operations has recently been completed by a consultant and offers guidance for stabilizing equipment, furniture and supplies (11).

(4) Priorities for identifying critical components, as in the case of structural components. Adequate design and construction is easily attained in new buildings but very difficult and expensive in existing buildings. Based on internal studies and studies by others (9) the VA is considering the priority guidelines in Figures 4 and 5.

It might be stated here that potential nonstructural hazards in existing buildings are identified and corrections of the most serious ones are included in the cost estimates. Others may be corrected over a period of time by Hospital employees.

In addition to these critical components, the VA has also been concerned with preserving the appearance of many of its buildings that are of significant historical interest. Historic preservation has been required by law only recently, although architectural and historical societies have had a keen interest for many years. Generally the law applies to important buildings fifty years old or older, but exceptions are often made. Many of the hospitals are on sites of former Army Posts, and therefore are an integral part of the history of that area of the United States. At the VA, we work closely with our Historic Preservation Officer who offers important guidance. This guidance is particularly important where one is considering exterior shear walls to reinforce a building's lateral force resisting system. Although not a completely reliable rule of thumb, the author can generalize that there is a distinct correlation; the weaker the mortar in an exterior wall, the more significant the structure from a historic point of view. Historic preservation requirements contributed to the very high unit cost of reinforcing some buildings.

COSTS OF REINFORCING EXISTING BUILDINGS

As was noted in the introduction, costs presented in this report were taken

FIGURE 4 - PRIORITY HOSPITAL NONSTRUCTURAL
FIXED SYSTEMS

FIRE PROTECTION SYSTEM

- Sprinkler System
  - Risers
  - Distribution mains
  - Check value to O/S distribution
- Standpipes
  - Mains
  - Risers
  - Pumps

HAZARDOUS MATERIALS

- Hazardous System (O_2 & N_2O)
  - Risers
  - Distribution mains
  - Hangers
- Hazardous Storage
  - Fuel (include Natural Gas)
  - O_2 cylinders/storage tank
  - Nitrous oxide

EMERGENCY POWER SYSTEM

- Transfer Switches
- Diesel-Generator
- Compressor
  - Fuel piping
  - Cooling system
    - Cooling tower
    - Pumps
    - Piping
  - Batteries
  - Controls
- Switchgear
- Substation
- Distribution Panels
- Motor Control Centers
- Panel Boards
- OR Isolating Panels
- Conduits and Bus

COMMUNICATIONS

- Telephone relay racks

TRANSPORT SYSTEMS

- Elevators
- Rails
- Counterweights
- Motors
- Generators
- Controls
- Hydraulic pumps

MECHANICAL SYSTEMS

- Heat Exchangers
- Pumps
- Absorbers
- Storage Tanks
- H & V Units above ceiling
- Compressors
- Cooling tower
- Graphic control panel
- Chiller
- Vacuum Pump
- Boiler
- Aerator
- Incinerator
- Controls
- Piping (over 4 " diam.)
  - Chilled water
  - Steam
  - Hangers
- Motors

MEDICAL SYSTEMS

- Stills
- Distilled water storage
- X-ray (overhead)

ARCHITECTURAL SYSTEMS

- Lighting Fixtures (anchors)
  - Emergency lighting/ batteries
  - Surgical
  - Stairwells
  - Ceilings
  - Corridor egress
    - OR, emergency
  - Partitions and Walls

FIGURE 5 - PRIORITY HOSPITAL FURNITURE
EQUIPMENT & SUPPLIES

FIRE PROTECTION SYSTEM

- Extinguishers
  - Receptacles
  - Mounting Brackets

HAZARDOUS MATERIALS

- Hazardous Storage
  - Radioactive storage
  - Chemicals, reagents
  - Anesthetic gases

COMMUNICATIONS

- Paging
- Alarms
- Radio
- PA Systems
- Nurse's Call
- Intercom Systems
- Program Systems
- Radio & TV Equipment-Amplifiers

MEDICAL SYSTEMS

- Fixed
  - Autoclaves
  - Film developers
  - Sequential multiple analyzer
- Portable
  - Free standing or wheels
  - Dialysis units
  - Appliances
  - Laboratory/medical equipment
  - Medical monitoring equipment
  - Beds, stretchers, carts, food service units
- Medical Stores and Supplies
  - Drugs and medications
  - Chemicals
  - Instruments
  - Linens
  - General supplies
- Medical Records

ARCHITECTURAL SYSTEMS

- Ornamentations
- Office Equipment
- Storage Racks, Bins, Lockers
- Operation Blocking Hazards
- Maintenance/Repair Shop Equipment and Tools
- Maintenance/Repair Stores and Supplies
  - Maintenance/repair parts
  - Housekeeping supplies
  - Emergency tools

OTHER EQUIPMENT

- Proximity to Critical Equipment
- Expensive Equipment
- Non-Emergency Power
- Sewer
- Kitchen Equipment
- Laundry Equipment

directly from consultants reports, but with no allowance for increased costs since the completion of those reports. Furthermore, the costs include allowances for correcting priority nonstructural items. Finally, the methods of correction were not necessarily the most economical because of VA's emphasis on minimizing the disruption to operations.

A series of histograms was presented in Figures 6 through 10 in an attempt to identify those factors that contribute to high costs. Figure 6 showed that the cost of reinforcing most buildings, including those housing patients 24-hours a day, is $10/square foot or less. Approximately 30 per-cent of all buildings can be corrected for $5/square foot or less.

Figure 7 showed that most buildings are two or less stories in height and have a floor area of less than 80,000 square feet. No major distinctions can be made relating reinforcing costs to either building height or area except that the average cost of reinforcing buildings over four stories in height seemed higher than for the other buildings.

Figure 8 showed a trend towards higher reinforcing costs where A_{max} is greater than 0.15 g. This trend would undoubtedly been emphasized if the older VA hospitals in California had been included in the study.

Figure 9 showed that the average cost of reinforcing masonry bearing wall construction is $5 to $10 per square foot, whether the floors are concrete or wood framing. A review of individual reports confirmed that none of the bearing wall structures are at high A_{max} sites. Steel and concrete frames with masonry infill walls are the most common type of construction in the VA system. The majority of buildings constructed since the 1920's have frames and infill walls. The spread in cost may be attibuted in large measure to the absence of a generally accepted metho of analysis for this type of building.

Figure 10 showed the costs related to various methods of reinforcing buildings Again, a review of the reports indicated that most of the diaphragm corrections were associated with masonry bearing walls. Otherwise, most buildings were corrected by adding either interior or exterior shear walls, or by adding both.

In his analysis of costs, Larrabee (7) reported that the cost of reinforcing Los Angeles schools and VA buildings were comparable, except that when the reinforcement cost of a school building exceeds 70% to 80% of the cost of a new building, the school is replaced. $30 per square foot is about the upper limit for reinforcement.

New VA Hospitals cost about $100 per square foot. However, these hospitals offter the finest hospital facilities and include features such as long span construction, interstitial space, sophisticated mechanical and electrical equipment, and so on. Because of the rapid advances of modern medicine, it is not possible for a layman to evaluate the level of obsolescence and the need for renovation of a particular hospital building. Thus at the VA, the engineers provide cost data. The Administrator decides, based on advice from his staff, on the proper course of action.

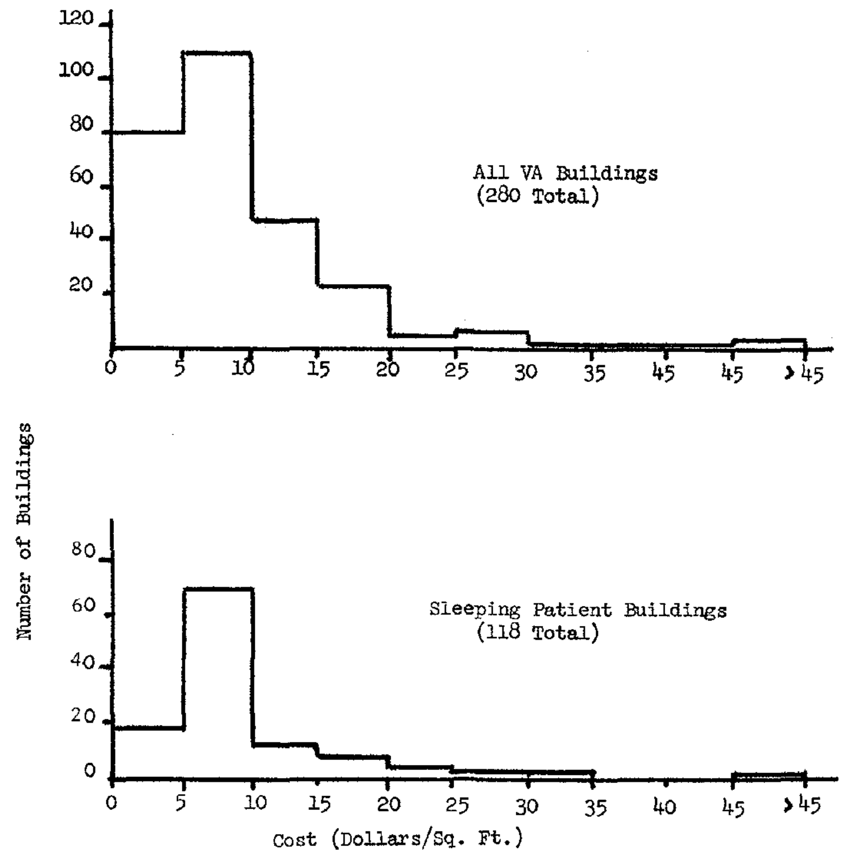


FIGURE 6 - REINFORCING COSTS FOR ALL BUILDINGS AND FOR SLEEPING PATIENT BUILDINGS

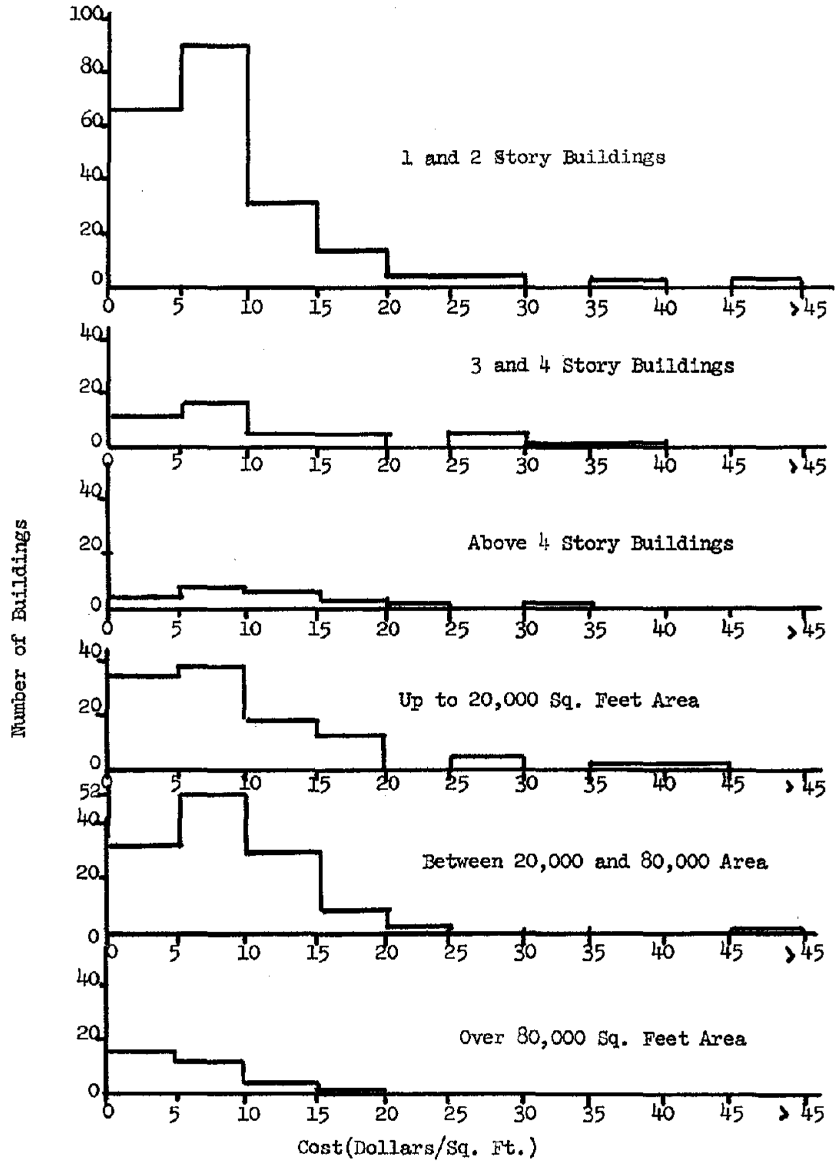


FIGURE 7 - REINFORCING COSTS BASED ON HEIGHT AND AREA

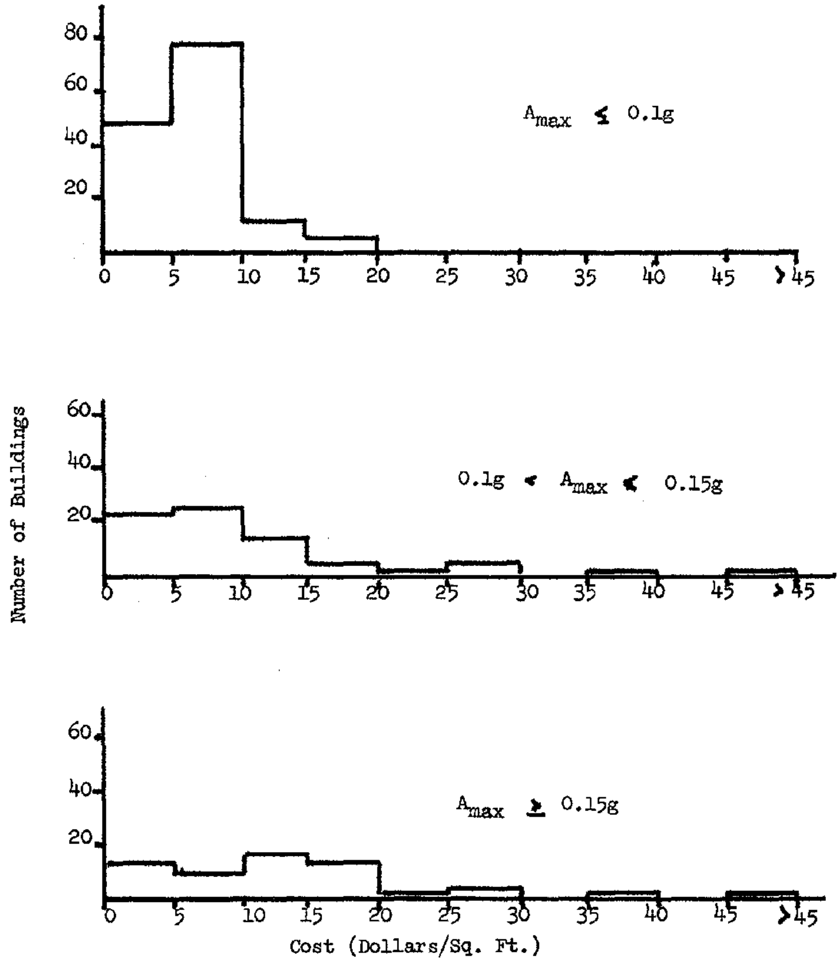


FIGURE 8 - REINFORCING COSTS BASED ON A_{MAX} (PEAK HORIZONTAL GROUND ACCELERATION)

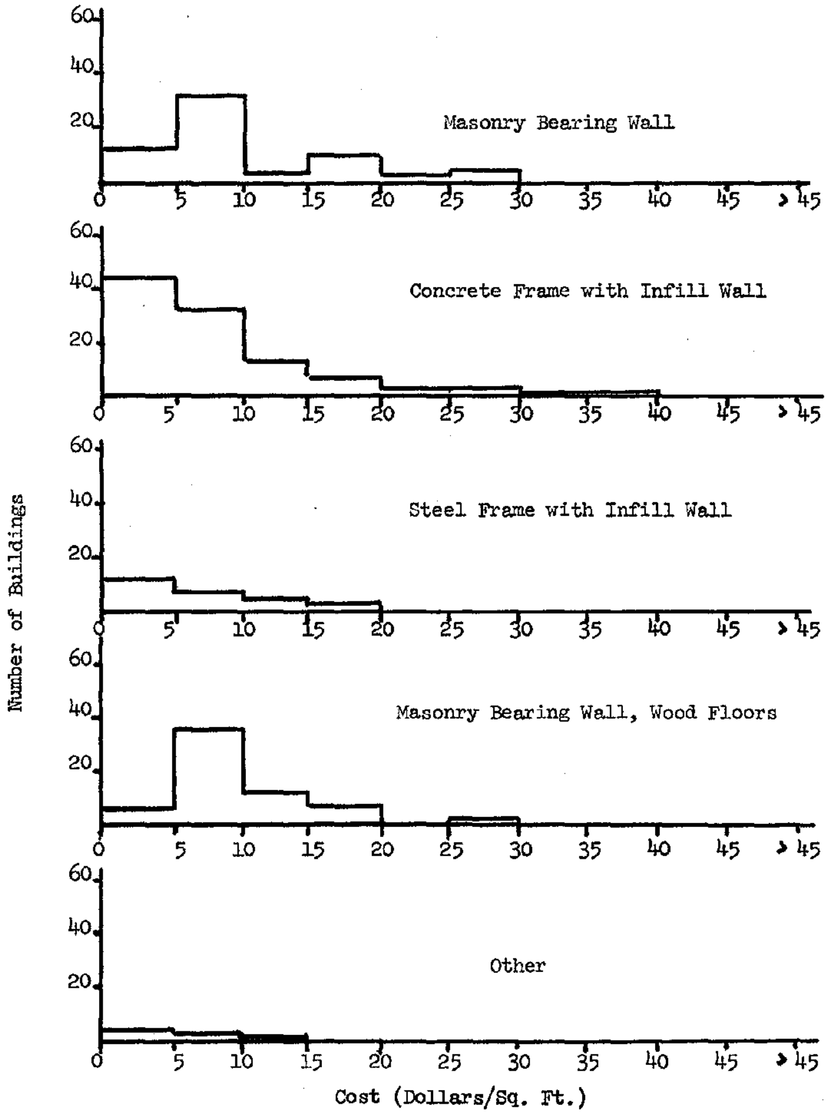


FIGURE 9 - REINFORCING COSTS BASED ON BUILDING TYPE

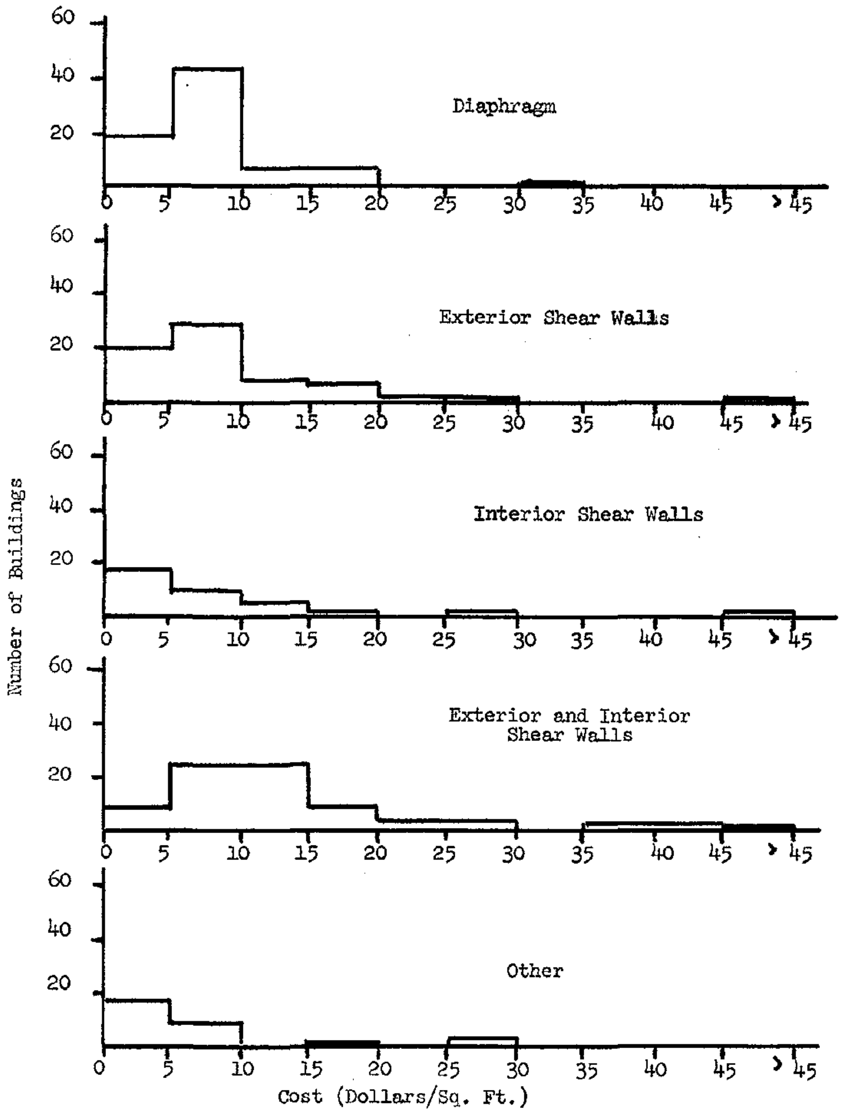


FIGURE 10 - REINFORCING COSTS BASED ON METHOD OF CORRECTION

SUMMARY AND CONCLUSIONS

Most of the participants in the Workshop are researchers who will discuss their latest findings in the field of Earthquake-Resistant Reinforced Concrete Building Construction. The author therefore decided to present a state-of-the-practice report based on the Veterans Administration Earthquake Engineering Program. The majority of buildings in the VA system are of reinforced concrete but it was believed that a report including all types of buildings would be more meaningful to the Workshop than one dealing with concrete buildings only.

Methods used by the author and consultants to analyze existing buildings were discussed briefly. A detailed look was taken at methods of reinforcing existing buildings and the relative costs associated therewith. Data was presented comparing reinforcing costs with height, area, ground acceleration, building type, and method of correction. A few distinctions were made based on the data, but the most definitive conclusions that can be drawn are that the cost of reinforcement is lower than is generally believed.

A major need in the field is for a generally acceptable method of analysis for existing buildings based upon research and actual performance in earthquakes.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED
CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

REPAIR AND STRENGTHENING OF REINFORCED CONCRETE MEMBERS AND BUILDINGS

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INTRODUCTION

The repair and strengthening of earthquake damaged buildings and the rehabilitation and strengthening of existing buildings require distinctly different considerations leading to appropriate techniques to accomplish the desired results. The degree of earthquake-resistant capacity that the structure should have in its rehabilitated condition is also a function of the political environment occurring after an earthquake or preceding an anticipated earthquake. A knowledge of structural and material properties of the existing structure must be evaluated in order to estimate its current strength and the strength of the structure following the completion of the suggested rehabilitation process. The steps necessary for implementation of this decision-making process and procedures for inspection and evaluation of existing hazardous or damaged buildings have been studied and reported, for example, by the Earthquake Engineering Research Institute and by the Applied Technology Council. In this paper it will be assumed that the existing characteristics of the structure can be determined or assumed and that the desired level of earthquake resistance has been decided.

The emphasis of this paper will be directed toward how our current knowledge of material and construction techniques can be used to design for increased earthquake resistance or restoration of the damaged structure without increased resistance. First a brief review of materials and procedures which could be used for repair and rehabilitation will be given and an evaluation of some of these materials and procedures will be summarized. The abridged bibliography is given to provide the interested reader access to many of the original sources. Finally the outstanding problems which need to be solved in order to assist the design engineers and building officials in evaluating the probable success of the proposed rehabilitation and/or repair will be discussed. It should be noted that a workshop sponsored by the National Science Foundation which is directed toward this problem will be held June 9-10, 1977. It is fortunate that the recommendations from that workshop will be available for our review here. Therefore the recommendations given in this preprint are expected to be modified before the workshop discussions on July 12, 1977.

REPAIR EXPERIENCE AND RESEARCH

Damage to reinforced concrete buildings and their members can be caused by a large number of different events. Some of the most common causes are differential settlement of the foundations, accidental dropping of heavy equipment or other objects, local overloads on the structure, explosions, corrosion or damage caused by spilling of chemicals, fire, and damage caused by severe winds or earthquake generated ground motion. Most types of this damage have been repaired by commercial firms and in many cases the repairs have been completed without consultation with the building official or the public. This is particularly true for buildings which are not open to the public. These commercial repair companies have the broad base of experience in utilizing various repair techniques and unfortunately much of this information has been held proprietary. However, some of their experiences can be used to study the repair/strengthening of earthquake damaged buildings and the rehabilitation of existing hazardous buildings.

Buildings damaged in Chile, Peru, Venezuela, Nicaragua, Guatemala, California, Alaska, and the Philippines give us many illustrations of what repairs and strengthening field procedures have been utilized. Only a few of these repaired buildings have been subjected to a second earthquake of similar or larger size for which the building had been repaired. In the few cases of repair/strengthened structures subjected to another earthquake the building was not significantly damaged in the second event. This is fortunate for the owner and designer but does not provide field experience with the overload characteristics of the strengthened building. Some post earthquake repairs have consisted of plastering over the apparent damage and repainting, some have used epoxy injection of cracked walls, beams and columns, and others have removed entire concrete beams, added additional longitudinal steel and stirrups and recast with new concrete. Sometimes the size of columns and beams have been increased, shear walls have been added, and in some of the most advanced repair/strengthening cases the designers were particularly careful to taper strengthened portions to provide a transitional change in strength and stiffness from the unchanged structure to the strongest, stiffest portions of the repaired structure.

The most important decision to be made regarding the repair or strengthening of a building is: What level of strength should the structure as a whole have? There are essentially three levels of repair/strengthening. The first is a replacement of the damaged material with new equivalent materials. This would return the structure to nearly the condition that existed prior to the damage. The second philosophy is to replace the damaged materials with stronger or additional materials to strengthen the damaged region. This would cause local strong spots and may also create locally stiff sections in the structural framing system. The consequences of these local changes must be carefully analyzed to determine their effects on the overall building behavior. The third option is to increase the strength and stiffness of the system. This option could be appropriate for hazardous as well as damaged buildings. In this last option we have the choice of either increasing the strength of the members of the existing structural system without a major change in the type of system or a second structural system can be added to the existing system. Again, the design of the additional structural system must be done carefully to produce a complete structural system that has the desired strength, stiffness, and ductility characteristics.

Characteristics of Repair Materials

Some of the more common materials which could be utilized for the repair of monolithic reinforced concrete construction and which have been used in the past for repairs are (a) epoxy-resins, manufactured by various companies, which can be used as a pressure grout to fill small cracks, (b) epoxy materials mixed with the various aggregates to fill large void spaces, (c) Portland cement concrete, Type III, this high-early strength concrete could be used in normal casting operations or for pneumatically applied mortar, (d) gypsum cement concrete, (e) quick-setting-cement concrete, which is a nonhydrous phosphate-magnesium cement concrete, and (f) fiber reinforced concrete. One common characteristic of most of these materials is their ability to achieve high strengths very rapidly. For repair and strengthening of existing buildings the speed of construction is more important than for original construction.

The connection of existing concrete with new materials will have to be considered. The anchorage of dowels, reinforcing bars, or various mechanical connections in existing concrete must be developed to insure the transfer of forces between the old and new materials. Utilization of prestressing to tie the elements together will not be discussed in this paper. Each of these materials is discussed with emphasis directed to their individual mechanical properties. The apparent advantages, disadvantages, and limitations on their use for various types of repair will be summarized.

(a) Epoxy-resins--The epoxy-resins are a manufactured chemical formulation which consists of an active and an inactive component which are mixed together prior to application as a bonding agent. The various chemical compositions of these agents are continually changing to meet new performance requirements. For structural purposes the material should contain 100% reactive solids. Detailed descriptions of the epoxy-resins and their recommended uses are available from manufacturer catalogs and will not be repeated here. A procedure for specification verification and quality control should be established for the selected material. In general the two epoxy components are mixed immediately prior to its application. The viscosity of the mixture and the time before the material hardens depends upon the characteristics of the agents being used. The most common of these resins used for building crack repair have been of the low viscosity type which means that they can be mixed and pressure injected into very small cracks. A reasonable time before hardening is needed so that the epoxy has time to completely fill the space between the cracked surfaces before setting. Upon hardening these materials have a very strong adhesion to the adjacent concrete and steel surfaces. Typical strength and stiffness properties of this type of epoxy have been given by Lee.

The higher viscosity epoxy-resin mixtures can be used for surface coating or for filling larger cracks or larger holes. In addition to the highly toxic nature of these chemicals it should be noted that the chemical reaction started by mixing the two components is exothermal. The heat generated by the chemical reaction can become excessive, causing a boiling of the mixture if too large a volume of material is confined without an appropriate way to absorb the generated heat.

The surfaces to which the epoxy mixture is applied must be clean. Presence of oil inhibits the adhesion of the material to concrete. Extremely fine concrete particles tend to form a surface which inhibits flow of the epoxy mixture through the cracks. It appears that clean breakage cracks are more easily repaired with the pressure grouting technique than would cracks that have experienced significant numbers of repeated cycles resulting in a grinding of the cracked surfaces resulting in a very fine powder. The deformations of reinforcing bars act similarly in pulverizing the adjacent concrete. It has been stated that cracks smaller than .003 cannot be effectively pressure injected. The epoxy mixture strength is dependent upon the temperature of curing as well as during load application. Temperature, time, and strength characteristics should be provided by the manufacturer.

(b) Epoxy-mortar--For larger void spaces, it is possible to combine the epoxy-resins, either the low viscosity or the higher viscosity materials as previously discussed with sand aggregates. This aggregate provides a heat sink to control the heat generation as well as provides an increased modulus of elasticity. For example, #4 sand used with a low viscosity epoxy results in about a five-fold increase in the modulus of elasticity.

The tensile strength of the epoxy-mortar is higher than the standard Portland cement concrete. The epoxy-mortar mixture has higher compressive strength, higher tension strength, greater shear capacity, and a lower modulus of elasticity than Portland cement concrete. Thus, epoxy mortar is not a compatible stiffness replacement material for reinforced concrete. The change in mechanical properties of epoxy mortar with large changes in temperature must be considered when a large volume of replacement material is used. It has been reported by Plecnik that the epoxy-concrete loses strength and stiffness rapidly under temperatures as low as 400°F. Therefore it must be fireproofed when utilized in a building. When the repair/strengthening is only for lateral load and not for gravity load conditions, fire is not as serious a problem because the probability of having a large earthquake occur shortly after a severe fire would be small.

(c) Portland cement concrete--Type III cement for making high-early strength concrete has been used for normal reinforced concrete placement and gunite application for many years. The properties of the concrete are described in the reinforced concrete literature and will not be repeated here.

(d) Gypsum cement mortar--Gypsum cement concrete has had rather limited use for structural application. Typical properties of structural gypsum cement mortar are given by Lee. There seems to be no advantage to use this material as a replacement for Portland cement concrete.

(e) Quick-setting cement mortar--This relatively new material is patented by Republic Steel Corporation and was originally developed for use as a repair material for reinforced concrete floors adjacent to steel blast furnaces. Need for a tough, strong material to carry heavy loads on steel wheels resulted in the development of this material. Its properties have been summarized by Lee. This material must be placed and cured in a water-free environment. The two components consist of a liquid and a dry aggregate and they can be mixed in a manner similar to Portland cement concrete. Before hardening is complete the tools can be cleaned with water.

(f) Fiber reinforced concrete--If it is desired to have a material that is stronger than the original material, particularly in the tension region, it is possible to use normal concrete or Type III Portland cement concrete as the basic material and add steel, glass, or plastic fibers in the mixing process. Properties of this material are given in the literature and will not be repeated here.

(g) Mechanical connections into existing concrete--The anchorage of dowels and reinforcing bars in existing concrete has had a limited amount of study. A common technique for providing the anchorage uses the following procedure. A hole larger than the bar is drilled. Epoxy, expansive cement, sulphur or other high strength grouting material is placed in the bottom of the hole and the bar is pushed into place and held until the grout has cured. At the present time the depth of embedment has been taken to be the same as ACI Code 318-71, Section 12.5 and field tests of these embedments have shown that the ultimate strength of the dowel or bar is developed. Additional tests have shown that a #10 bar embedded 10 diameters is able to develop its yield strength without pull-out. In order to develop criteria for a repair procedure, additional experimental data should be developed to determine what reasonable minimum embedment can be used with epoxy or high strength grout materials to develop the tensile strength of the bar.

Mechanical types of anchors use wedging action to provide anchorage. The manufacturers of these mechanical connectors have specific recommendations for installation and the strengths that can be developed. A limited number of proprietary tests on the cyclic load characteristics of these anchors have been made. A great deal more information is required in order to make specific recommendations for the use of these type of anchors in earthquake-resistant modifications.

Techniques for Repair to Original Strength

The decision of which repair technique to utilize for a given situation depends upon the degree of damage and the level of repair/strengthening to be accomplished. First we will assume that the level of repair is to replace the damaged material in order to restore the nominal original strength at the time of the damage. Further it will be assumed that no special inspection procedures are used; only individual observation with some minor mechanical probing will be used to decide the degree of damage. The repair techniques will be described below in groups according to the observed level of damage. Evaluation of the success of repair reflects somewhat upon the success of damage observation.

Damage level - cracks only--If the earthquake has caused cracking in the concrete and the cracks are reasonably small (opening width of less than 1/4 inch), the repair technique used to develop the strength of the reinforced concrete element has been pressure injection of epoxy. A procedure for epoxy injection of cracks is as follows:

First the external surfaces are cleaned of non-structural surface materials so that the concrete surface is open. Then plastic injection ports are placed along the surface of the crack on both sides of the member and are secured in place with an epoxy sealant. The center-to-center spacing of these ports should be between one and 1.2 times the thickness of the concrete element. However, the spacing is dependent upon the width of the element and whether or not pressure injection will be accomplished from both or only one side of the member. After these ports are in place the surface of the crack between the ports is sealed with epoxy sealant.

After the sealant has cured low viscosity epoxy grout is injected into one port at a time beginning at the lowest point of the crack in case of a vertical crack, or at one end of the crack in the case of a horizontal crack. Working at this port, the epoxy is pressure injected until a bleeding of the epoxy material is viewed from the opposite side of the member at the corresponding port or from the port next higher on the same side of the member. When this flow is seen, the injection port is closed and the injection equipment is moved to this next port. As each port is closed the pressure injection moves to the successive ports above or along the crack. After all of the ports are completed the final port is closed and all ports remain closed until the epoxy along the entire crack has cured. This is normally a two-man operation with epoxy injection occurring from one side.

The smaller cracks require higher pressure or more closely spaced ports to obtain complete penetration of the epoxy material throughout the depth and width of the member. Larger cracks will permit larger port spacing, dependent upon the width of the member. This technique is appropriate for all types of structural elements - beams, columns, walls and floor units. The behavior of members repaired by this technique in the laboratory and subjected to load conditions similar to the original damaging conditions has shown that the failure occurs adjacent to the epoxy repairs. In other words, the repair material is stronger than the adjacent concrete material. However, the failure mechanism for the structure has not changed because the new failure is in the concrete adjacent to the previous failures.

Two other items should be noted. First, if there is a loss of bond between the reinforcing bar and the concrete through a number of cycles of deformation, the concrete adjacent to the bar has been pulverized to a very fine powder. This fine powder effectively dams the epoxy from saturating this region. It should not be expected that pressure injection of the cracks will restore the bond of the concrete to the reinforcing steel for this condition. This and the fact that not all of the small cracks will be epoxy injected results in a structural system which will be less stiff than the original system. The decrease in stiffness is the consequence of the small cracks that are unrepaired and the lost bond between the reinforcing steel and the existing concrete.

It is appropriate to consider whether this is adequate repair. It has been shown that the original strength can be developed, although it has been recommended that only 70 to 80 percent of the original strength recovery be assumed because of the possibility of lack of penetration of all the cracks in the section. Because the member was damaged it is probable that the original section may not provide the sufficient strength for the structure. Therefore, a technique for strengthening these elements to avoid a similar type of damage during the next earthquake should be considered.

Large cracks and crushed concrete--For cracks larger than 1/4 inch or regions in which the concrete has crushed treatment other than epoxy injection is required. The loose concrete should be removed, leaving only solid concrete. The material that has been removed can be replaced with new material. A replacement of the damaged concrete can be made with either an expansive cement mortar, Type III high-early strength mortar, or other material. The selection of the replacement material depends upon the desired repair characteristics as described in the section on characteristics of repair materials. The existing material characteristics must be determined before the repair material is selected. Since the damage was rather severe, consideration should be given to the need for additional shear reinforcement or flexural reinforcement in the repair region. This can be easily accomplished if the reinforcement does not have to be developed into the adjacent solid concrete regions. What this does is make the repair section stronger than the adjacent existing material and the subsequent failure in the following earthquake would probably occur adjacent to this repaired region.

In the case of wall and floor diaphragm damage it may be more economical to add new steel on the outside of the wall surface and thicken the wall surface by an application of gunite concrete rather than trying to repair the damage to it. This will be discussed in the section on increased stiffness or strength. The increased weight of the structure caused by adding new materials must be considered in the re-analysis of the building forces and the foundation pressures.

Fractured, excessively yielded or buckled reinforcement--In the repair of severely damaged reinforced concrete members it is possible that the reinforcement has buckled, experienced elongation with excessive yielding, or in the extreme case, may have even fractured. This reinforcement can be repaired by replacement with new steel, using butt welding, lap welding, or in some cases by a splice. If practical the repair should be made without removal of the existing steel. The best approach depends upon the amount of space available in the original member. It is recommended that additional confinement steel be added to delay future buckling of the bars in this region. This additional steel will not substantially increase the strength of the member, however it will extend its inelastic strength carrying ability.

Evaluation of these techniques--The evaluation of the success of a specific repair process is a very difficult problem. The determination of the original damage was recognized earlier as being inaccurate because inspection techniques utilize visual observation of damage. Based upon these visual observations a specific repair technique was selected. For example, if the concrete appeared to have a limited number of cracks without a great amount of cross cracking resulting in loose pieces of concrete the decision probably would have been to epoxy inject the larger of these cracks and the minor cracks would be left unrepaired. The epoxy repair by injection can be evaluated by taking a core of the concrete material and observing the amount of crack not filled with epoxy material. However, this is not recommended for reinforced members. From a selected number of samples the success of the injection repairing of existing cracks can be judged. However it is impossible to fill very small cracks and the cyclic breakdown of the bond between reinforcing steel and concrete does not appear to be successfully repaired by this injection process. Therefore the structural system will be less stiff than the original system and depending upon the length of bond breakdown the structural system could also be weaker than the original system. On the basis of experiences in the San Fernando earthquake, the City of Los Angeles has recommended that only a 70 percent recovery of original strength by the epoxy injection process be assumed. This appears to be a reasonable judgment for situations where the damge is clear.

Next consider the case of more substantial damage where original material must be removed and replaced with new repair material. The strength of the existing material must be determined. Then the repair material can be selected and the reinforcement repaired as discussed. Where stronger materials have been added as a replacement for the original concrete and more confinement stirrups were added around the reinforcing steel, the repaired section will be significantly stronger than the original and subsequent damage will occur outside this repaired section. Either the adjacent portion of the beam or the connection will be damaged if the beam is repaired. Where the connection was repaired by the introduction of stronger material, the failure could occur either in the columns or the adjacent beams. In recent studies by Lee of the repair of beam-column subassemblages it was found that when the original failure was in the connection the repair of this region by stronger materials resulted in a shifting of the failure into the adjacent beam with relatively stable force-deformation moment rotation characteristics as expected of a concrete frame meeting the current ACI/UBC ductile moment frame reinforcement conditions. In cases where the non-seismic moment frame design was used the original failure was in the beam. The beam end material was replaced with stronger material and the connection area was not repaired because of the very minor cracking which could be observed. Retests of these repaired subassemblages showed a very rapid loss of strength of the subassemblage and severe damage of the connection. At the present stage of development it appears to be nearly impossible to predict when the connection will not be adequate for the repaired system. Without question the inelastic deformation should be made to occur in the beams or at least in the column sections and not within the connection itself when not designed for ductile moment conditions. Since failure of the connection forms a hinge joint which uncouples the intersecting members, the stiffness will decrease very rapidly.

It is clear that additional research will have to be undertaken in order to determine the effect of confinement caused by beams and floor slabs framing in from all directions to the connection region. An interior connection may have damage that cannot be seen because of its location, or the connection may be in good condition due to the confinement provided by the connecting members. It is recommended that when beam ends are severely damaged a significant amount of the concrete from the connection area be replaced in the process of repairing the beam ends. Stronger material should be placed as far as practical into the connection region as well as into the damaged beam end region. The consequence of this could be that the next failure would occur in the columns rather than in the adjacent sections of the beam. The overall behavior of the repaired structural system will have to be evaluated to determine the consequences of having these columns fail.

If the damage is sufficiently widespread throughout the building, it seems practical to strengthen the system so that in future earthquakes the damage of the structure can be minimized or eliminated.

Techniques for Repair and/or Strengthening to Increased Strength and/or Stiffness

If the decision to strengthen or stiffen the building in the process of repair has been made, a thorough analysis and design of the structural framing systems should be made during the process of selecting the best technique for accomplishing the desired increases. The level to which the system will be strengthened or stiffened must have been made according to established criteria. The techniques for accomplishing the desired changes can be classified into two distinct groups. The first would consist of complete removal and replacement by a totally new structural system. The second option would be the addition of a second structural system which would act together with the existing structural system to accomplish the overall desired strength and stiffness. With each of these options the existing load conditions of the structure must be considered at the time of the strengthening. The effect of the additional structural system must be considered on the behavior of the superstructure and the foundations of the building. The decision as to the best type of structural system to use, moment frame system or shear walls at specific locations, has to be made with the same criteria as for an original building. However, the design is more complex because the local effect of adding elements must be considered in the overall response of the system.

In the case of removal and replacement by a new system there is little to be added here to our current construction techniques. Particular care must be exercised in tying the horizontal floor diaphragms into the lateral force resisting system. Even if the lateral-vertical load carrying system has increased strength, if the connections to the horizontal diaphragms are not adequate, subsequent failures will occur in this junction between the stiffened system and the existing floor diaphragms. The techniques discussed above for repair and addition of new materials to damaged regions is applicable to junction between materials which remain and the new structural system.

The addition of new structural systems to the existing system provides a more difficult problem. The existing internal stress condition of the members which are being strengthened must be considered in analyzing the behavior during subsequent overloads caused by the anticipated earthquakes. Care must be taken to provide a smooth transition of stiffness and strength in the structural system just as we would require on any new construction.

Columns and beams--Increases in column size and girder sizes can be accomplished by adding additional reinforcement adjacent to the existing columns, providing sufficient confinement of steel and tying the system together as with a new construction. Where the existing columns have adequate longitudinal reinforcement but insufficient ties, Sasaki has shown that column encasement with rectangular or circular steel sections or with steel straps and then grouted to fill any voids provides the desired shear capacity of the member. This provides increased ductility for these members. For members expected to fail in shear, this repair/strengthening procedure was particularly beneficial.

Walls--The most efficient way to increase the stiffness and strength of a building is to provide additional shear walls or to thicken the existing walls. Tso, Gyoten, Higashi, Kahn and Plecnik have all provided data on various forms of incorporating new wall material. Existing shear walls can be strengthened by supplying new steel on the outside of the existing wall and increasing the wall thickness with additional concrete usually by the gunite process. Care must be taken to anchor the ends of the horizontal and vertical reinforcement into the adjacent columns and beams in order to provide the necessary continuity.

Both precast and cast-in-place walls have been studied. Both systems were tied into the existing beams and columns with dowels or mechanical anchors. Wing walls and partial opening shear walls were studied. The most important aspect of these studies which still requires additional research is the development of reasonable ductility of the shear walls. The slitted shear wall originated by Muto is one example of what can be done. I expect that other sessions of this workshop will be discussing this problem. Development of the wall reinforcement into the boundary elements also needs study.

Evaluation of techniques--With good material and field control techniques it may be possible to develop repaired structural characteristics consistent with the design assumptions. It is hoped that the repair/strengthening can be evaluated by standard testing methods using samples of the material utilized in the construction process. All of this seems reasonable based upon the laboratory experiments reported to date. In order to provide the necessary validation of these results to actual construction will require experimental studies of actual field repairs.

PROBLEMS REMAINING

The most significant problems to be solved is the focus of a workshop sponsored by the National Science Foundation to be held June 9-10, 1977. The results of that workshop will form the basis for the discussion here. As a preliminary summary of the outstanding problems to be solved in the repair/strengthening of reinforced concrete buildings the following are provided.

1. One of the most important areas that needs research at the present time is damage assessment. Realistic field inspection techniques in addition to visual techniques are needed to estimate the degree of damage or deterioration of the existing building in terms of potential loss in strength and stiffness of the elements. This is particularly true in the case of connection regions which cannot be observed because of framing of additional members from the transverse directions. The effect of small cracks on the overall strength and stiffness characteristics of the structural system need further identification. Some research has been done on the loss in stiffness caused by small cracking. In the case of nuclear power reactors a correlation between loss in shear strength and the width of cracks has been investigated. It is hoped that research on real structures can be initiated in order to determine the loss of stiffness and strength of a structural system with various degrees of cracking and damage. The development of instrumental techniques for locating and evaluating damage regions deserves a great deal of study. Potential benefits of such instrumental inspection techniques are clear.

2. Verification that the proposed and laboratory evaluated repair/strengthening materials and construction techniques are accomplished in real buildings. It is suggested that a limited number of real buildings need to be repaired/strengthened and tested to failure.

3. Additional data is needed on the development of dowels and mechanical anchors subjected to cyclic loading at damage levels.

4. Since new epoxy materials are continually being developed, a testing standard for both short and long term loading determination of their mechanical properties is needed. The consequences of these mechanical properties on the effectiveness of the repair/strengthening procedure must be continuously evaluated.

5. Additional research is needed to determine the properties of precast elements introduced into existing structural systems and the properties of their connections. Both the connection of precast elements to existing concrete members and to other precast elements need more development for earthquake loading conditions.

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METHODS OF STRUCTURAL ANALYSIS

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED
CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

THE ART OF MODELING BUILDINGS FOR DYNAMIC SEISMIC ANALYSIS

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INTRODUCTION

During the last 15 years, the art of modeling buildings for dynamic seismic analysis has advanced from one-dimensional to full three-dimensional mathematical idealizations. The major factor contributing to this rapid advancement in the art has been the development of computer programs with a wide selection of versatile element routines, efficient equation solvers, and vertually unlimited problem size. These computer programs have not only provided the means for developing sophisticated mathematical models, but also served as the research tool in verifying and refining the modeling techniques.

Even with the advanced computer technology we enjoy today, modeling of buildings is still more an art than a science. Experience, engineering intuition, and good assumptions all play a major role in the development of models that produce realistic results.

This paper will attempt to present the major consideration in developing a dynamic model such as stiffness, mass, damping, and the geometric distribution of each through the structure. It will attempt to identify the degree of uncertainty associated with each idealizing assumption and probable influence on the end results of the analysis performed with the model. Further, it will identify the areas of research and development needed to improve the state-of-the-art from the standpoint first, of a structural engineer and second, of a structural researcher.

CONSIDERATIONS IN SELECTION OF MODEL TYPE

The selection of the mathematical model and its degree of complexity or refinement is dependent on the following considerations:

1. The complexity of the structure,
2. The accuracy with which physical parameters used in the model or analysis are known,
3. The type of response to be determined or the information required from the analysis, and
4. The accuracy of the solution being sought.

Buildings with highly irregular geometry, complex structural framing schemes, or complicated nonstructural (architectural or functional) requirements may require three-dimensional consideration in modeling and analysis in order to capture the significant dynamic response.

The accuracy to which input parameters are known will also affect the level of model complexity. For example, assume structural mass is estimated to within 20 percent of actual conditions, stiffness of concrete members (gross versus cracked section properties) is estimated to within 50 percent of actual conditions, damping is estimated to within 50 percent of actual conditions, and the earthquake ground motion is estimated to within 100 percent of actual conditions. How significant is the added complex of a three-dimensional model under these conditions of uncertainty if it provides only a 30 percent improvement in accuracy over that of a two-dimensional model? The general rule followed in design practice is to construct a model and perform calculations to an accuracy consistent with the input parameters. Here is one of the major motivations for research to improve and refine the model input data.

The model type and complexity will also vary depending on the end results desired from the analysis. A very simple "stick" type model may be used to represent a structure if only gross response in terms of story forces, shears, and overturning moments is desired. On the other extreme, a three-dimensional space frame supporting delicate emergency equipment vital to the function of the facility after an earthquake would probably require a full three-dimensional model with time history response analysis in order to accurately compute the structural member forces and the motions at equipment support points. Thus, information required from the analysis will directly influence the type and complexity of the model, as well as the type of dynamic analysis (i.e., response spectrum, modal time history, or direct integration time history).

The model type selected and its degree of complexity and/or refinement may be dependent on the level of accuracy sought in the solution. If benefits can be obtained from increased accuracy, then model refinement or increased complexity is reasonable. Benefits may include increased structural reliability, increased confidence in a given level of reliability, or perhaps reduction in construction cost. For vital facilities such as hospitals, and civic nerve centers, which must remain functional after an earthquake, the level of structural reliability or capacity of the structure to survive a given earthquake is of major importance. Current equivalent static lateral force design procedures do not provide a direct estimate of this capacity as demonstrated by recent earthquakes such as San Fernando.[1,2]

Nonlinear analysis with models possessing inelastic post-yield force deflection relationships have been used to establish the ultimate earthquake collapse resistance of structures.[3,4,5] These same modeling procedures have been applied in the design of vital facilities under Collapse Threshold earthquakes to establish the level of reserve capacity present in the design. Significant cost savings were possible through the use of these refined models when the ductility of the structural system was directly taken into account in resisting the Collapse Threshold event inelastically.[6]

BASIC MODEL TYPES

There is a wide range of mathematical models that may be used to represent flexible structures such as buildings. The commonly used models (shown schematically on Figures 1 through 4)[7] include:

1. Simple one-dimensional cantilever beam models
2. Two-dimensional frame and shear wall models

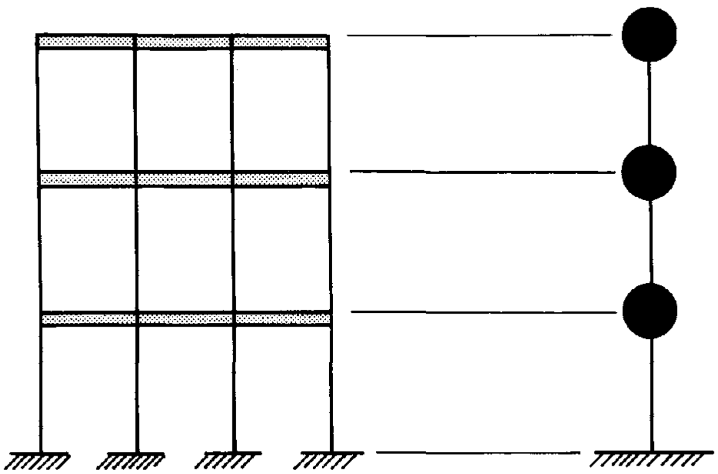


FIGURE 1a FIXED BASE

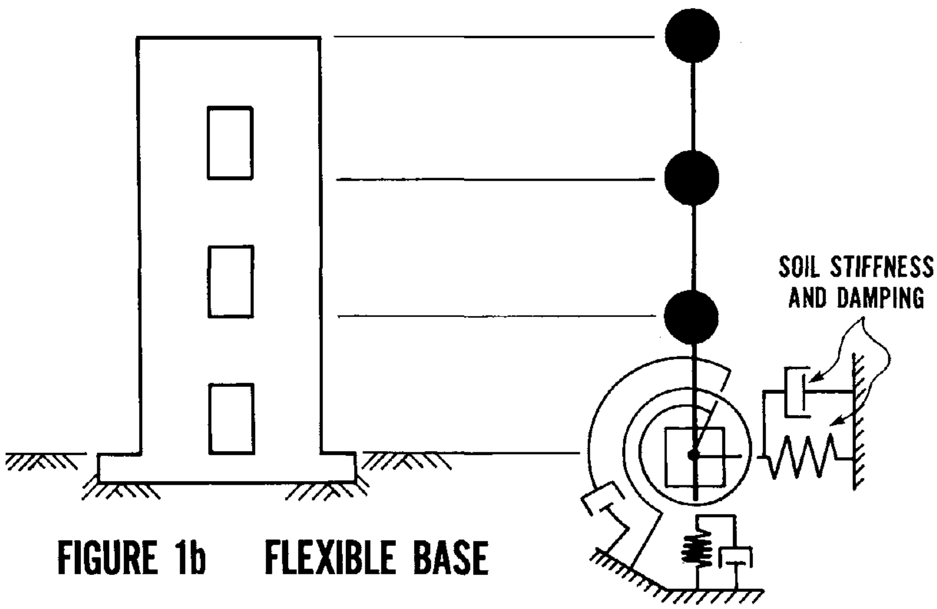


FIGURE 1b FLEXIBLE BASE

ONE–DIMENSIONAL CANTILEVER BEAM MODEL

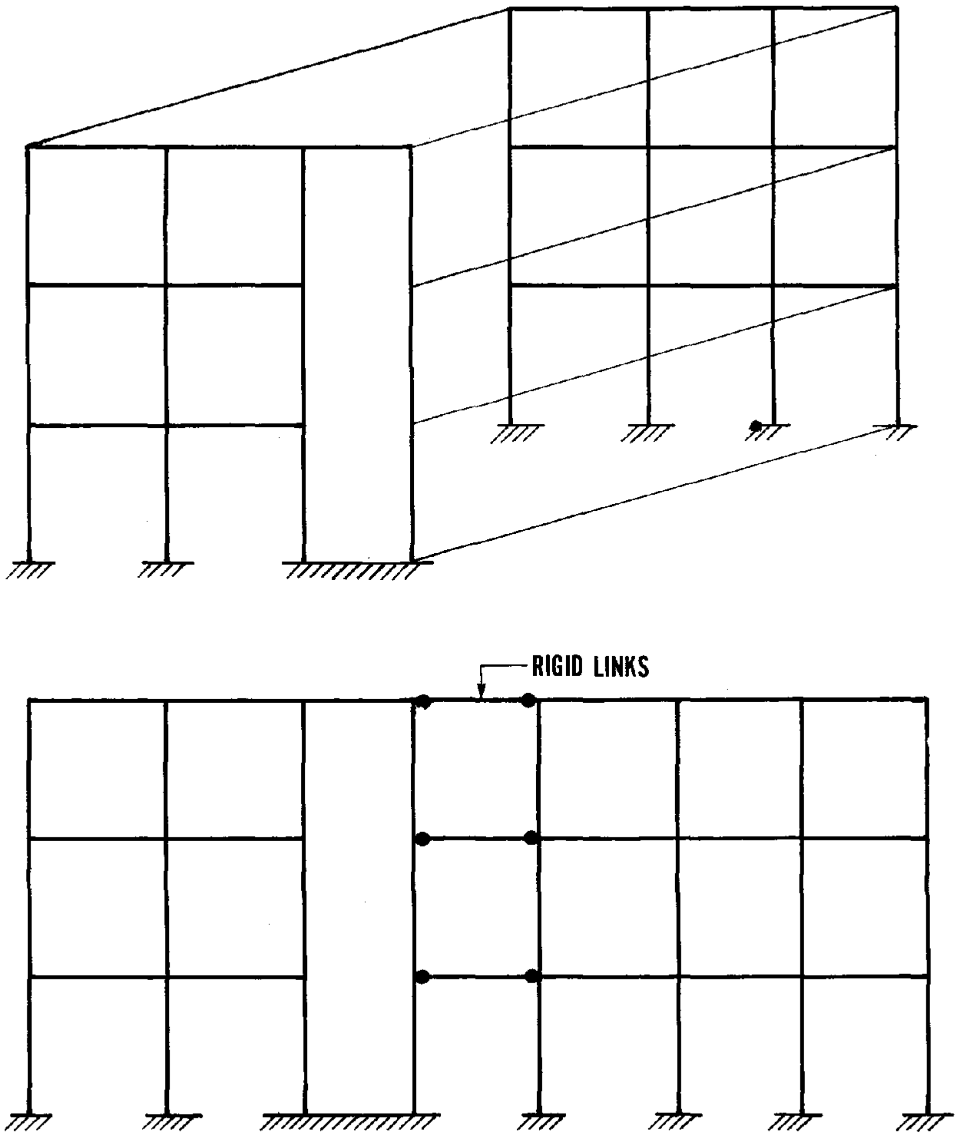


FIGURE 2
TWO–DIMENSIONAL FRAME AND SHEARWALL MODEL

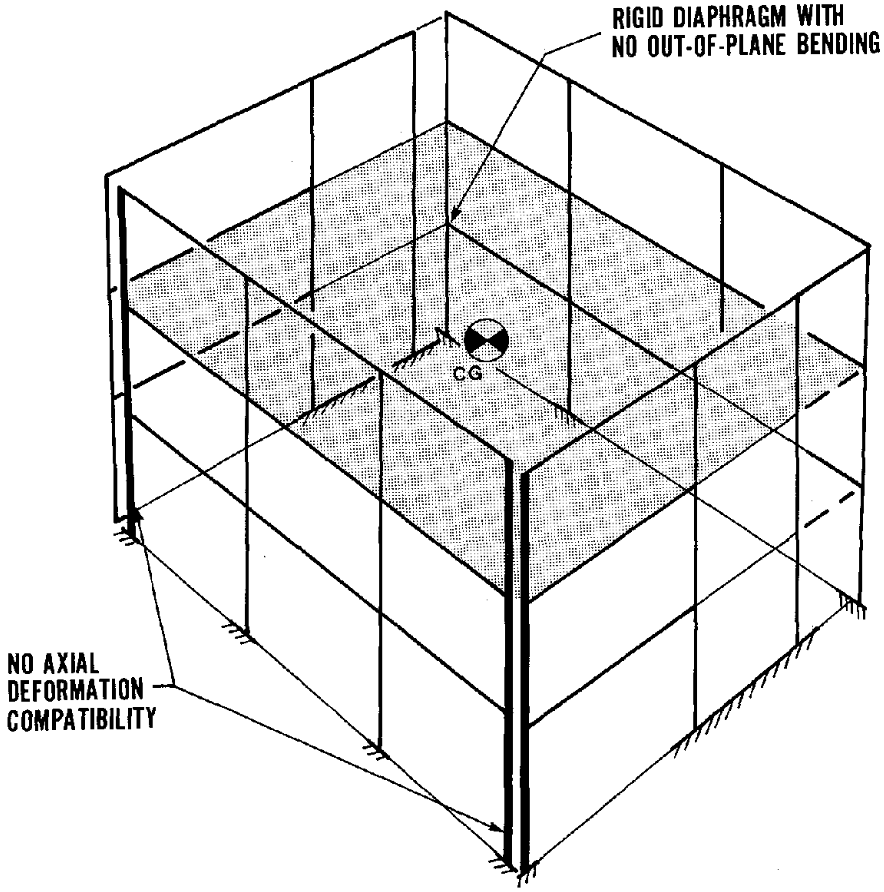


FIGURE 3
PSEUDO THREE–DIMENSIONAL BUILDING MODEL

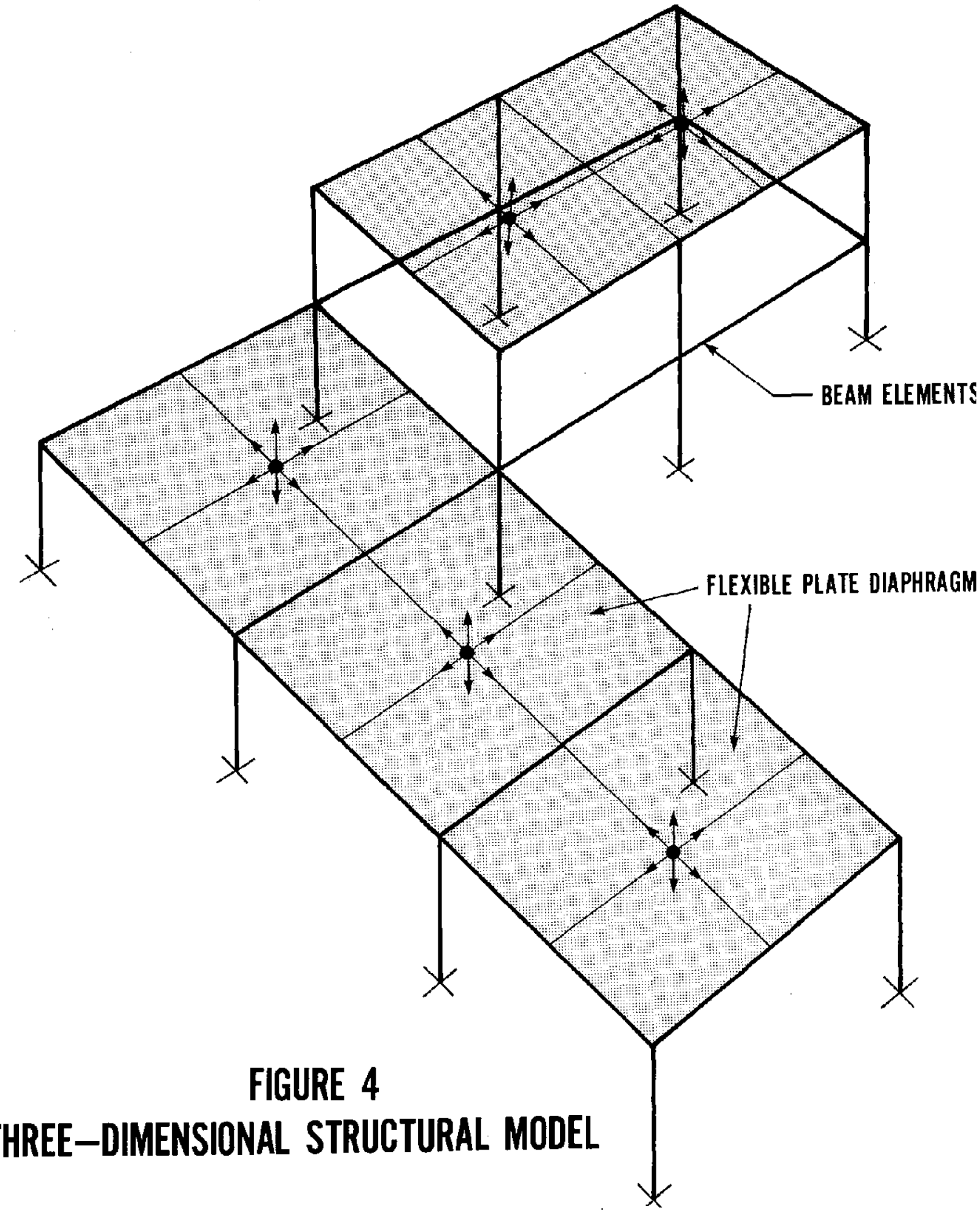


FIGURE 4
THREE–DIMENSIONAL STRUCTURAL MODEL

3. Psuedo three-dimensional building models
4. Three-dimensional structural models.

These models may be further subdivided into classes based on the physical properties of mass, stiffness, and damping. For most design purposes, the mass is lumped at each story rather than distributed through the structure. The stiffness is distributed and treated elastically. The damping is treated as velocity proportional and expressed in terms of a critical damping ratio. This class of model is normally referred to as an "elastic" model or as a lumped mass, distributed stiffness model.

The other model class is the inelastic or nonlinear model. In this case structural stiffness of the members is dependent upon a prescribed nonlinear force deflection algorithm. Geometric nonlinearity may also be included in this class of model to account for change in stiffness due to large deflections. Damping is treated as mass and stiffness proportional. The distinction between these two classes of models will be specifically noted throughout this paper.

One-Dimensional Cantilever Beam Model

The cantilever beam is the simplest of all the flexible building idealizations. It is commonly referred to as a "stick" model in the literature. The total lateral stiffness of the structure is represented by the flexural and shear stiffness of the vertical beam, while the total building mass at each floor level is lumped at appropriate heights along the weightless beam as shown on Figure 1(a). For firm soil conditions, the cantilever is assumed rigidly attached to the ground. For soft soil sites, soil-structure interaction may be modeled by transitional and rotational springs attached to the base of the cantilever along with a parallel set of dashpots to represent material and radiation damping effects (see Figure 1b).

This simple one-dimensional structural model may be analyzed by hand as well as by computer to determine its period, mode shapes, participation factors, and modal response. Lateral story forces, shears, overturning moments, deflections, drifts, and accelerations may be computed.

A major drawback with this model is the added analytical work in converting story forces and shears into member forces for design purposes. The story forces or shears must be distributed to each level and each member on a modal basis (i.e., mode-by-mode), and a modal summation performed to compute the member forces. This modal force distribution process may be performed manually, or by a static analysis computer program.

A second drawback of the one-dimensional model is that it cannot be readily adapted to represent inelastic structural behavior except in simple structural cases such as a single shear wall system or soil stiffness nonlinearity. In all other cases where multiple member inelasticity is possible at a given level in the structure, the single beam idealization cannot provide direct member design forces. For inelastic analysis, the laws of superposition no longer apply.

Two-Dimensional Frame and Shear Wall Model

The two-dimensional model of frames or frame-shear walls (see Figure 2(a)) is used to idealize a regular building having little torsional eccentricity. All the frames and shear walls in one direction are treated as planar systems tied together at each floor level by rigid links which represent the floor diaphragm (see Figure 2(b)). It is assumed that the floor diaphragms are essentially rigid.

For structures with floor diaphragms that are very flexible relative to the shear walls or frames, modeling is simplified by uncoupling each plane of frames or frame-shear walls and performing separate solutions. Tributary floor areas are used to compute the mass assigned to each frame at each floor level. Multiple dynamic analyses would be performed, one for each plane of frames or shear walls.

The key advantage of the two-dimensional frame and shear wall model over the simple cantilever beam model is the direct input of structural member properties and direct computation of member forces. There is no need to calculate the equivalent cantilever beam stiffness properties or distribute story forces to the various columns, beams, and shear walls based on relative member stiffness. The model can also be used effectively for nonlinear analysis as well as elastic analysis.

As a disadvantage, semirigid floor diaphragms cannot be adequately represented in the planar frame or shear wall model. A three-dimensional model is generally required. The solution is also too complex for hand analysis.

Psuedo Three-Dimensional Building Model

The most commonly used structural model today is the psuedo three-dimensional model shown on Figure 3. The model consists of an assemblage of two-dimensional frames and shear walls that may be arbitrarily oriented in plan. The frames are connected at each floor level by a diaphragm which is assumed rigid in its own plane. The diaphragm is generally assumed to have no out-of-plane flexural stiffness.

The model may have three dynamic degrees of freedom at each floor, two translation and one rotation about the vertical axis. Two translational mass quantities and a torsional mass moment of inertia are computed for each floor as well as the plan location of the center of mass. Dynamic torsional response is directly computed in this analysis when all three mass quantities and their locations are used.

The major advantage of the psuedo three-dimensional building model over the two-dimensional frame and shear wall model is the direct computation of torsional response. This model is ideal for buildings with highly irregular plan dimensions resulting in large eccentricities between center of mass and stiffness.

The model has been referred to as "psuedo" three-dimensional because vertical motion is not modeled. Only the two horizontal components of ground motion are considered directly in the analysis. In addition to this limitation, most of the computer programs that solve this problem such as TABS and

XTABS[8,9] neglect compatibility of axial deformation in columns common to more than one frame. This is generally not a severe limitation, except for tall slender buildings or tube-type structures. The ETABS[10] program has incorporated a compatible axial deformation option for common columns.

Further refinement on the psuedo three-dimensional model is required for buildings with semirigid diaphragms, large open court areas, and mezzanine levels that are discontinuous between frames permitting two story, laterally unsupported, column lengths. One program not commercially available treats floor diaphragms as a series of rigid plates (subdiaphragms) connected by flexible links. This permits modeling of diaphragms with large openings, or two-building towers connected by floor diaphragms at one or more levels.

Computer programs currently used to perform the psuedo three-dimensional analysis with elastic models include: TABS, XTABS, and ETABS. Inelastic analysis may be performed with the program DRAIN-TABS[11] which is about to be released through the NISEE/Computer Application Department of University of California at Berkeley.

Three-Dimensional Structural Model

In a full three-dimensional structural model, all beams, columns, shear walls, and floor diaphragms are represented as three-dimensional members in three-dimensional space. Each structural joint has six degrees of freedom, three translational and three rotational. Structural mass may be lumped at each joint throughout the structure in a distributed manner, or it may be lumped at a few key joints.

A three-dimensional model will respond to vertical as well as horizontal components of ground motion. Torsion and rocking motion of the structure is computed along with translation. However, torsional and rocking ground motion components are not commonly used as input motions to the analysis.

The primary advantages of the three-dimensional model is its ability to represent an arbitrary structural geometry without restriction of member orientation and its ability to model virtually all members such as flexible diaphragms without imposing restrictive assumptions such as rigidity or extreme flexibility.

The chief disadvantage of the three-dimensional model is the time involved in developing and checking the model, which is generally considerably more than required to develop the other types of models. The computer costs for running this type of model is also greater than the simpler models such as the psuedo three-dimensional.

General purpose computer programs used commercially to perform the three-dimensional analysis on elastic models include: SAP IV,[12] NASTRAN,[13] STRUDL-DYNAL,[14] EASE,[15] and STARDYNE.[16] There are a few general purpose programs capable of performing three-dimensional, direct integration, time history analyses with nonlinear models; these include: NONSAP,[17] ANSR-1,[18] ADINA,[19] and NASTRAN.

STIFFNESS REPRESENTATION

The total building stiffness is a composite of the stiffnesses from structural members, such as frames, shear walls, and floor diaphragms, and from nonstructural elements such as partitions and from the supporting foundation. To further complicate the problem of modeling, the relative contribution of structural, nonstructural, and foundation stiffness to the total model varies with the level of motion (e.g., distortion) produced by the earthquake. Under small ambient vibrations produced by micro tremors, all elements in the building remain essentially elastic, and nonstructural joints and connections between partitions and the structure function together to form an integral force resisting system. Under these conditions both the structural and nonstructural elements contribute stiffness to the building. Under large amplitude motions produced by a major earthquake, the accidental ties between structural and nonstructural elements may be partially or totally broken or the nonstructural elements may become damaged resulting in a loss of stiffness contributed by these elements. Finally, under very large amplitude motions, not only may the nonstructural elements be damaged, but there may be yielding of structural elements as well, resulting in considerable loss of stiffness and potential collapse.

A similar softening or reduction in stiffness takes place in most soils during major earthquakes due to large strains or distortion produced by the seismic waves as they travel through the soil medium. Soil stiffness reduction is highly dependent upon the dynamic stress-strain properties of the particular soil. As a rough guide the reduction in stiffness may range from 50 to 100 percent depending on the amplitude of motion and soil characteristics.

Thus, the amplitude of motion is a major consideration in establishing the appropriate method for modeling of building stiffness. As an example, the accelergraph records from high rise buildings in the 1971 San Fernando earthquake when compared with pre- and post-ambient vibration survey data on the same structures showed a significant lengthening in fundamental building period and associated reduction in stiffness.[1] For buildings with reinforced concrete frames as the primary lateral force resisting system, the fundamental period lengthened by 70 percent[20] and the stiffness was reduced to one-third of its ambient vibration value. This earthquake stiffness matched that of the elastic structural frame for the buildings that were mathematically modeled such as the Muir Medical Center.[1] All nonstructural stiffness was apparently lost. For the Holiday Inns at Marengo and Orion and the Bank of California, loss of stiffness from nonstructural partitions was not sufficient to explain the 200 to 300 percent lengthening in fundamental period.[1] In these buildings, yielding and inelastic deformation developed in many of the beams and beam column connections.

For buildings with reinforced concrete shear walls as the primary lateral force resisting system, the fundamental period lengthened 30 percent,[20] during the earthquake with an associated 40 percent reduction in stiffness. Some cracking was observed in the lower level shear walls of the buildings, such as the Ceritified Life Building.[1]

To reiterate, the amplitude of building motion is a major consideration in the selection of appropriate model stiffness properties. There is no single ideal model for reinforced concrete buildings that is applicable over the full range of motion amplitudes from ambient vibration to extreme earthquake. From a research standpoint, the closest thing to the ideal solution would be an inelastic structural model which consists of nonstructural partition elements that provide stiffness under small amplitude vibrations, possibly up to one-fourth the yield limit of the structural system. At larger strains or deflections, the partition stiffness would degrade rapidly to zero. The structural system would be in parallel with the nonstructural elements, such as frames or frame walls or shear walls. To model elastic behavior of the structural system at deflections less than those producing gross member inelasticity, a variable modulus of elasticity might be used to approximate the softening characteristic of concrete as micro cracks develop and expand to form fully cracked section properties. In essence the elastic concrete structure is really inelastic, even at strains less than yield. As flexural members go beyond yield and plastic hinges form or as walls crack and deform in shear, the stiffness of each structural element must be modeled individually and its history of inelasticity kept on record. Ultimately, when sufficient numbers of large amplitude inelastic excursions have occurred, the member stiffness would degrade and the member would be eliminated from the structure. Flexural members such as beams might loose all moment resistance in the process of degradation.

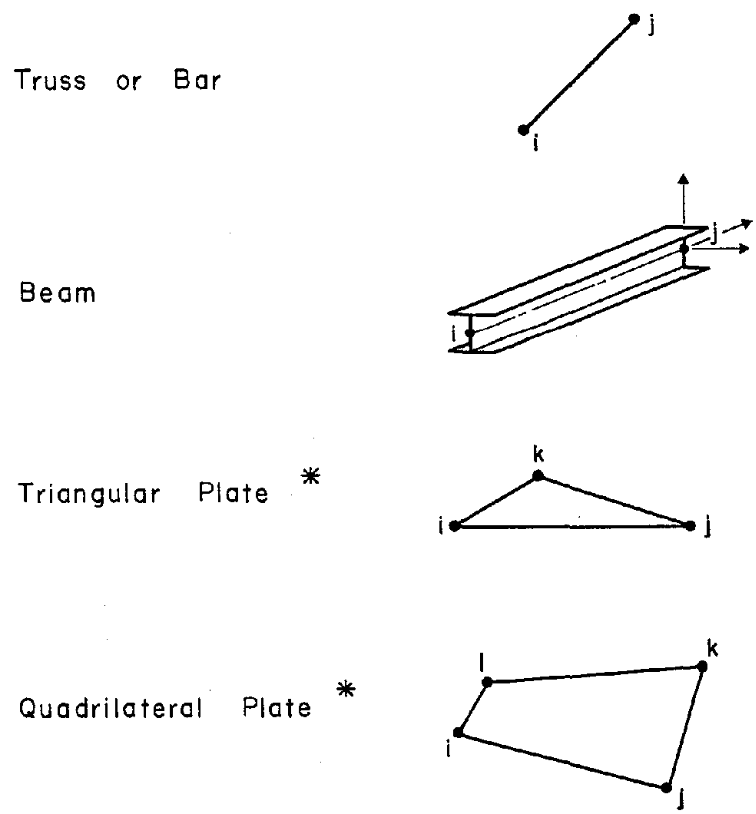
Such an idealized model sounds like a dream (nightmare) to the practicing structural engineer who is restrained by cost and time limitation to model the structure in the simplest way possible. However, the nonlinear model described is within the current state of the art and offers a potentially useful research tool. In fact, nonlinear soil models have been developed which permit detailed study of inelastic soil-structure behavior.

Structural Member Stiffness

There are several finite element types that are commonly used to represent the stiffness characteristics of structural elements in buildings. These are beam elements and truss elements. Plate elements are also used, though less prevalently. The elements depicted on Figure 5 are all available in most general purpose structural analysis programs.

Truss Elements--Truss or bar elements are used most frequently to model structural elements such as struts, truss members, unidirectional springs, and bracing system components. Occasionally it is convenient or necessary to model a more complex physical system with an assemblage of truss elements. For example, a model using truss members may be used to represent in-plane plate or membrane behavior of a floor diaphragm, shear web, or shear wall.

The truss element is generally modeled with linear elastic stiffness properties. Tests on axially loaded columns have generally shown compression members to possess little ductility. Thus, it is common practice to design these members to behave elastically. There is limited theoretical or experimental work on stiffness degradation of cyclically loaded bar elements in the post yield and buckling range.[21]



* Plate elements may be for use in two dimensional problems, such as plane stress (membrane) or plane strain, or in three dimensional problems, such as for plate bending or thin shell applications.

TYPICAL FINITE ELEMENTS
FIGURE 5

Ref. 7

Beam Elements--The simplest beam element is one that considers bending and shear stiffness about only one axis. However, the more general beam elements available in many computer programs model shear and bending behavior about each principal axis, as well as axial stiffness and torsion. Beam elements in models usually correspond one to one with actual beams and columns in structural frames. However, they may also be used to synthesize the stiffness of more complex systems. A typical application is a series of beams used in a stick model (see Figure 1) to represent an entire building, one beam per story or level. Beam elements with shear distortion capability are often used to model a shear wall or slab for in-plane bending or shear. Sometimes an array or grillage of beams may be used to represent the transverse bending behavior of walls or slabs.

Post elastic flexural stiffness properties of beam elements have been determined from dynamic cyclic tests.[22] Computer programs are available for modeling the moment-curvature behavior of two-dimensional beams including cyclic degradation [23] (i.e., loss of strength and stiffness with subsequent cycles of inelastic flexural deformation, as shown on Figure 6). Most ductile-moment-resisting frames are designed under the assumption that plastic hinges will form in the beams under large earthquake motions. However, no hinging or inelastic axial deformation is normally permitted in the columns, which are usually modeled with the same beam elements.

Plate Elements--The triangular and quadrilateral plate elements are used in the psuedo three-dimensional and the general purpose three-dimensional computer programs. The in-plane or membrane stiffness of the plate element is used in modeling shear walls and flexible floor diaphragms. The plate bending stiffness is occasionally used to model out-of-plane deformations in floor slabs and walls.

Some programs provide a shear panel element capable of modeling only the in-plane shear stiffness of walls or diaphragms. There are both elastic[12] and inelastic[23] versions of the shear panel. The element in DRAIN-2D[23] possesses an elastic-plastic, strain-hardening, force-deflection algorithm with a brittle failure mechanism at a prescribed deflection, as shown on Figure 7. This model does not possess a stiffness degradation algorithm which would permit closer approximation with current research.[24]

Section Properties

The softening effects produced by crack propagation in reinforced concrete under earthquake strain have already been noted. Since the full member length is seldom stressed to cracking, it is common practice in modeling to work with gross (uncracked) section properties. These are generally simpler to compute, and provide a shorter period building, which normally results in higher design force levels under current seismic codes. Cracked section properties provide a lower limit on elastic member stiffness.

Test data from reinforced concrete frames stressed to just below yield on the shaking table at the University of California at Berkeley have shown that the flexural stiffness of members could be approximated by using eight-tenth (0.8) the gross section properties. On the other hand, reinforced concrete frame structures such as the Muir Medical Center and Sheraton Universal

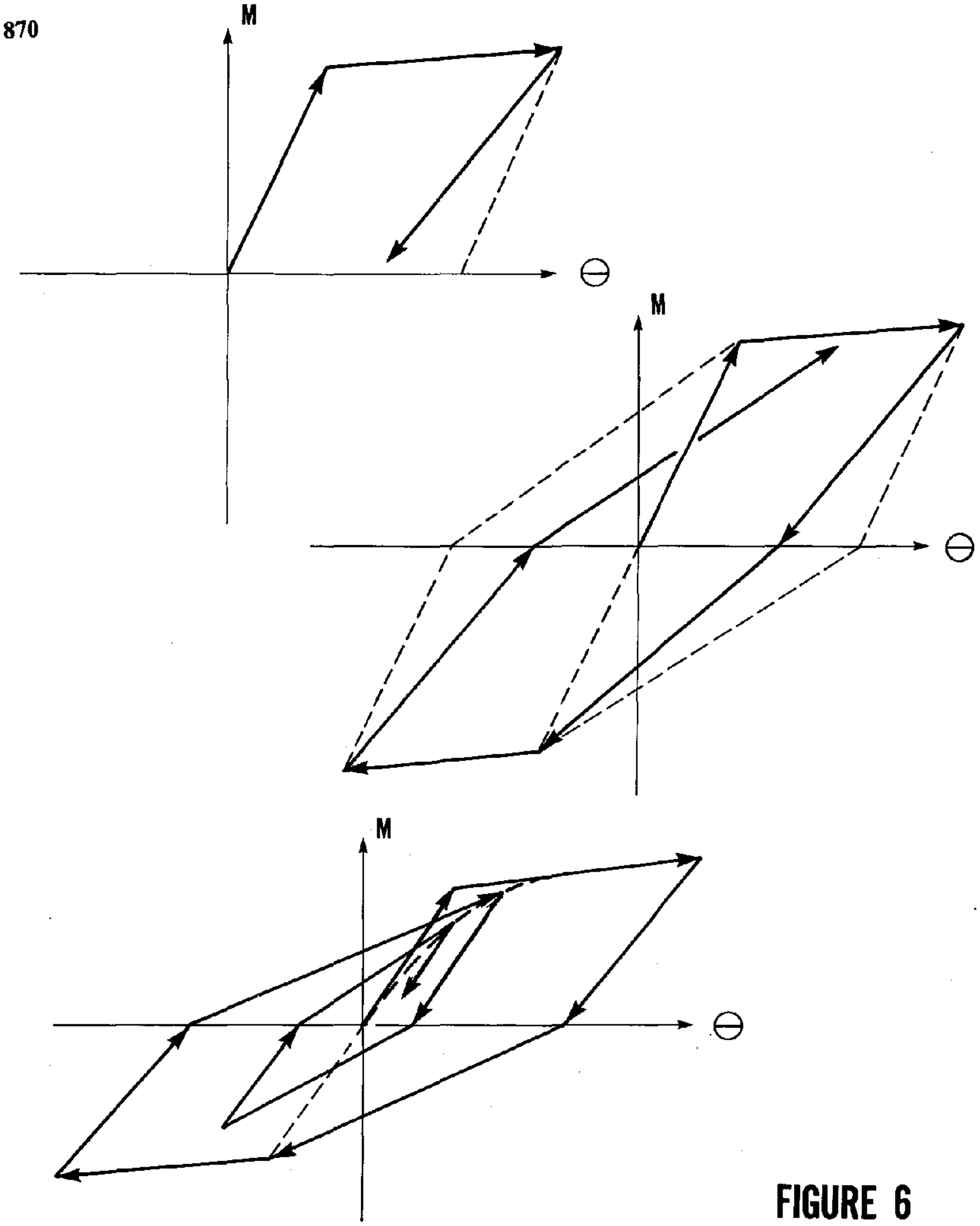


FIGURE 6
MOMENT–ROTATION ALGORITHM
WITH DEGRADING STIFFNESS

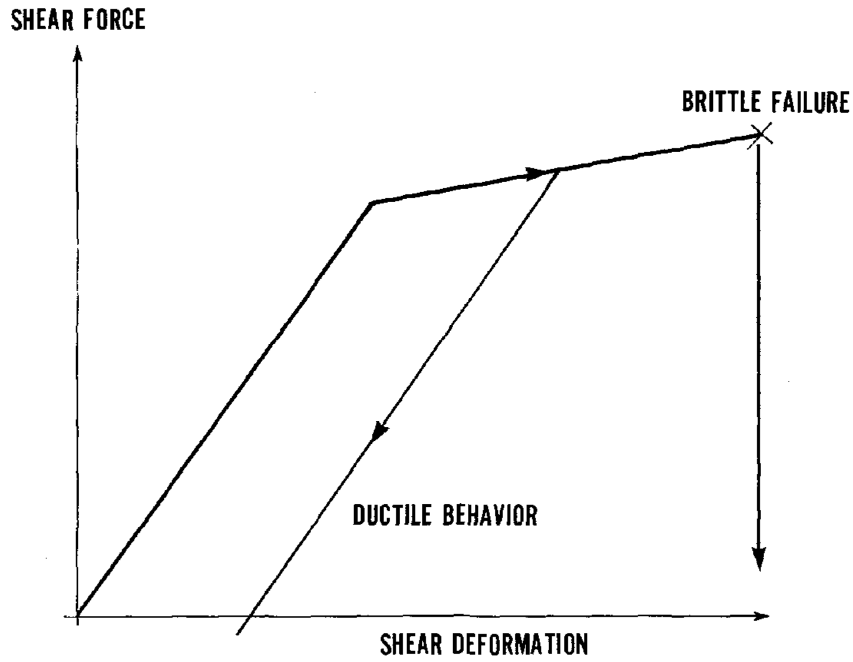


FIGURE 7
FORCE–DEFORMATION ALGORITHM
FOR SHEAR PANEL

Hotel[1] when modeled with gross section properties provided good agreement with San Fernando earthquake data. Computed force levels in these structures were above code design level but below yield.

When determining the stiffness properties of concrete T-beams or L-shaped spandrel beams assumptions must be made about the effective cross sectional area. The width of slab, which directly contributes stiffness to the beam, is a variable due to shear lag. Under dynamic response some engineers believe the effective section to be greater than the limiting flange widths specified by the ACI code. Others feel the effective section should be less than specified by ACI because reversal in moments may take place in the beam at the column supports during the earthquake.

The question of shear lag and effective flange width is equally applicable to shear walls that intersect in a common corner. Typical examples are elevator and stair towers where all the walls are integrally tied together by the lacing action of the stairs and floor diaphragms. Much judgment must be exercised by the engineer in these cases. Some treat each wall as independent, neglecting all flange stiffness. Others include flange stiffness, per ACI limits on flange width. Some tie all the walls together in a single box to compute section properties. In certain instances the truss-like action of the stair risers is included in the stiffness calculation.

In developing the stiffness properties for individual members, it is common practice to work with minimum concrete strengths at 28 days as the criterion for deriving the modulus of elasticity. We know from concrete test specimens that the strength at 28 days is going to be well above the specified minimum (or else!). Furthermore, with age, the stiffness increases by 20 to 30 percent. Thus, the modulus of elasticity during the major portion of the structures life could be 30 to 50 percent or greater than the code minimum specified for design.

There is a counter influence in certain environments that reduces the effective stiffness. It is the commonly ignored corrosive attack from environmental conditions which cause the concrete to crack, reinforcing steel to corrode, and concrete cover to spall away, reducing the effective section and strength of the members. Classic examples are the concrete structures in oil refineries next to the sea. The combination of salt water mist and airborne chemical effluents can age and deteriorate concrete in a short order.

<u>Clear Span or Finite Joint Size</u>

For slender beams and columns with relatively large span-to-column width (or beam width) ratios, the use of centerline dimensions in modeling member lengths is generally adequate. However, in most reinforced concrete structures, the column and beam widths may produce a significant increase in structure stiffness. Most designers try to incorporate the column width in the analysis to take advantage of reduced design moments at the face of the column rather than using the higher values computed at the centerline.

For reinforced concrete frames it is common practice to assume the joint to be infinitely rigid. Computer programs such as TAB, XTABS, and ETABS[8,9,10] formulate the member stiffness based on the rigid joint assumption. For programs that do not directly accommodate the finite rigid joint, it is possible to model the member with an equivalent stiffness property incorporating the rigid joint zone. The Portland Cement Association[25] has developed tables of member properties for beams with rigid end links for use by the designer in this situation.

As an alternative, most general purpose computer programs may be used to directly model the rigid joint by insertion of an additional member at the joint. This member is given a very large stiffness value, say 100 to 1,000 times the value of the flexible connecting beam or column. Some programs have rigid link elements in the program library for this application.[12]

Floor Diaphragms--Floor diaphragms are one of the most overlooked and oversimplified elements of a structure. They range in stiffness from rigid to very flexible depending on their geometry and their relative stiffness to that of the lateral supporting system. Diaphragm deflection limits and span/depth limits have been defined in the Tri Services manual on "Seismic Design for Buildings."[26] This guideline for classifying diaphragm web stiffness serves as a starting point for selecting appropriate modeling assumptions.

For rigid diaphragms all the lateral force resisting elements at the floor are assumed to deflect laterally the same amount (i.e., for a symmetrical building without torsion). For flexible diaphragms, it is assumed that each lateral supporting element of the building deflects independently without diaphragm intertie. Semirigid and semiflexible diaphragms are those that have significant lateral deflection, but also have sufficient stiffness to distribute a portion of the load to the lateral force resisting elements in proportion to their relative rigidities. The behavior of the semirigid diaphragm is analogous to a continuous beam on elastic supports. The support reactions are dependent on the relative stiffness of both diaphragm and lateral force resisting elements.

In modeling the rigid diaphragm, all lateral force resisting systems are assumed to deflect identically. Rigid links may be used to tie the system together at each floor,[12] or the computer program may directly impose this constraint on all frames and shear walls.[8,9,10] The flexible diaphragm is modeled by neglecting all force transfer between lateral force resisting elements and allowing each to move independently.

Modeling of the semirigid and semiflexible diaphragm is more time consuming. The actual stiffness and mass distribution along the diaphragm must be represented by equivalent beams or plate elements. Generally, a three-dimensional representation of the structure is required in this case. Most engineers would prefer to use a simplifying approximation and bound the problem. Typically, the semirigid diaphragm would be assumed rigid or the semiflexible one would be assumed very flexible and eliminated altogether.

The author has studied the dynamic response of semirigid diaphragms modeled with the various simplifying assumptions as well as by the rigorous idealization. The results of the study (see Figure 8) showed convincingly that the simplified models could not conservatively bound the dynamic support reactions computed from the more rigorous model of the floor diaphragm.

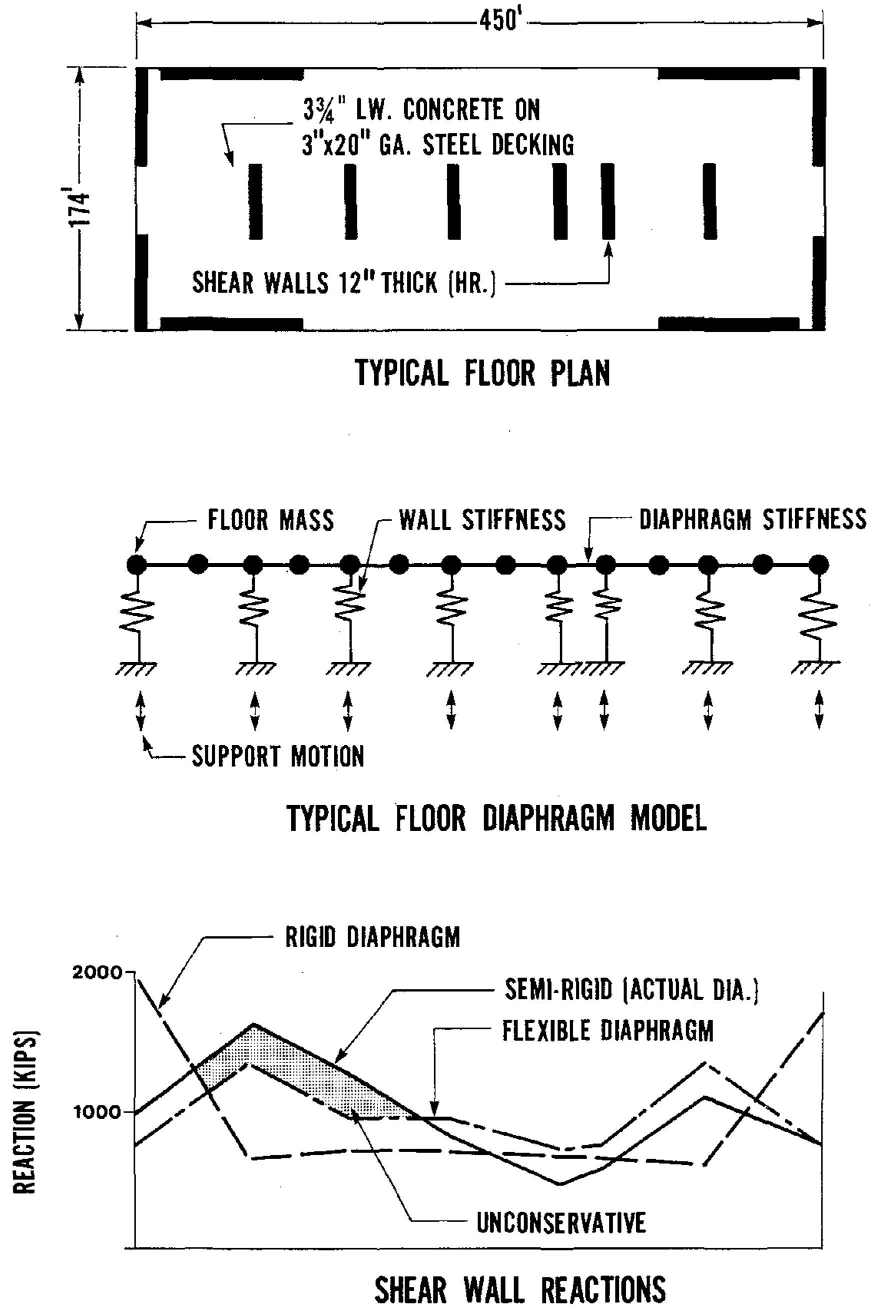


FIGURE 8 DIAPHRAGM FLEXIBILITY STUDY

For long narrow diaphrams, perforated diaphragms, and those of unusual geometry such as "L's" and "T"'s, special studies should be performed to assure appropriate diaphragm modeling before the full structural model is developed. Ramped floors in parking structures serve as diaphragms and may also act as a truss system. It is common practice to ignore the stiffening effect from floor diaphragm truss action in modeling parking structures. The significance of this effect on design needs further study.

Nonstructural Elements

Walls, partitions, stair risers, etc., which are not part of the structural system, can still participate in the overall structural stiffness. Normally, these nonstructural elements are ignored when developing the stiffness properties for the structural analysis. The City of Los Angeles currently requires that all major nonstructural frames or poured-in-place partitions be included in structural models when computing the fundamental building period. However, the nonstructural stiffness and strength is excluded from the model when it comes time to distribute the design forces.

MASS REPRESENTATION

Distributed mass properties can be mathematically represented as continuous quantities in a continuous media. However, in discrete coordinate problems, mass properties must be discretized. Mass may be either lumped at the nodes in some arbitrary fashion, or the distributed mass within individual finite elements may be computed as a lumped mass using the consistent mass method.

Lumped Mass

In the lumped mass representation, inertia properties are concentrated at the nodes. These inertia properties may be translational masses or rotary inertias if rotational degrees-of-freedom are considered in the dynamic model. The connecting elements and space between nodes are assumed to be massless which means that any mass associated with the elements is assumed to move effectively as a rigid body with the mass points (degrees-of-freedom) at assigned nodes.

The method of lumping masses so that the dynamic behavior of the model accurately represents the continuous real structure is as much an art as a science. In buildings, the mass of floors and walls is usually assigned to appropriate dynamic degrees of freedom on a tributary area basis. In one- or two-dimensional models, the entire floor mass is lumped at the center of gravity of the floor plane. In these cases it is generally necessary to assign a rotary inertia to the torsional degree-of-freedom in order to calculate the torsional response of the structure.

All permanent weight that moves with the structure is lumped at the appropriate nodes. This weight includes the dead loads of structural elements; architectural systems, such as ceilings, facades, partitions; and mechanical systems, such as piping, equipment, etc. To account for possible live loads present in the structure, it is common to include 20 percent of the design live load as part of the computed mass property.[1,7,27] For warehouse structures, all of the live load is included in the structural mass.

Torsional Mass

To establish whether torsional response will be significant and should be represented in the structural model, most engineers use the torsional eccentricity in the building as a guide. This is the computed distance between center-of-mass and center-of-rigidity or center of translational stiffness. If the eccentricity is less than 5 percent of the plan dimension of the building in the direction of eccentricity, torsional response may be arbitrarily neglected by the engineer. For eccentricities over 5 percent, torsional effects generally will be included in the model. For eccentricities 10 percent or greater, torsion should definitely be modeled.

There are situations where torsional effects may be significant, even though the computed torsional eccentricity is very small (less than 5 percent). If the natural period for the torsional mode is close to the translational mode, the two modes of response can reinforce each other resulting in large torsional response. From a practical viewpoint, it is wise to include the torsional effects in the model of any major structure in which a dynamic analysis is performed.

Symmetrical structures with large live load masses such as warehouses represent an interesting study in torsional effects. If the structure could be partially loaded in a highly unsymmetrical live load placement pattern, torsional effects should be design considerations.

Rotational Inertia

Rotational inertia mass of a building floor about a horizontal axis is normally neglected in conventional building design. The author has found the effect of rotational mass on building periods to be negligible. However, for inelastic structural models, the presence of large rotational inertia masses at the member joints can have a very marked influence on the post-yield response of the structure.

DAMPING

Energy dissipation in the form of damping is normally idealized in linear elastic dynamic analyses as viscous or velocity proportional for convenience of solution. Physically, damping in structures may be a combination of structural damping and Coulombic friction. As an indication of normally accepted values for material damping, the partial list from Newmark and Rosenblueth, page 422, is reproduced in Table 1. Damping is highly strain dependent as well as the material dependent. The damping values in Table 1 are applicable at various strain levels in the structural members up to yield stress. Beyond yield, hysteretic damping becomes a major source of energy dissipation and should be modeled directly with a nonlinear force deflection algorithm.

TABLE 1

TYPICAL VALUES OF DAMPING
(After Newmark and Hall)[28]

Stress Level	Type and Condition of Structure	Percentage of Critical Damping
1. Low, well below proportional limit, stresses below 1/4 yield point	Steel, reinforced or prestressed concrete, wood; no cracking; no joint slip	0.5-1.0
2. Working stress, no more than about 1/2 yield point	Welded steel, prestressed concrete, well reinforced concrete (only slight cracking)	2
	Reinforced concrete with considerable cracking	3-5
3. At or just below yield point	Welded steel, prestressed concrete (without complete loss in prestress)	5
	Reinforced concrete and prestressed concrete	7-10
4. Beyond yield point, with permanent strain greater than yield point limit strain	Reinforced concrete and prestressed concrete	10-15
5. All ranges; rocking of entire structure*	On rock, v_s >1800 m/sec	2-5
	On firm soil, $v_s \geq 600$ m/sec	5-7
	On soft soil, v_s <600 m/sec	7-10

*Higher damping ratios for lower values of shear wave velocity v_s.

Nonstructural elements, such as masonry partition walls, provide a significant amount of damping which is normally neglected in the modeling assumptions.

FOUNDATION MODELING

Ground motion records used for seismic analyses are usually considered free field motions, i.e., free from the presence of structures. Studies have shown that the presence of a normal structure (such as a frame structure) on firm soil does not significantly modify the ground motion at or adjacent to its base. However, the dynamic response of a stiff, massive structure supported on relatively soft soil may significantly alter the ground motions from those in the free field.

The effects of soil-structure interaction become more important in reducing building periods and modifying dynamic response for heavy structures which are stiff relative to the soil on which they are founded. For example, massive rigid buildings with shear walls or braced frames on soft soils will be good candidates for showing significant effects from soil-structure interaction. The Certified Life Building, a shear wall structure in which strong motions were recorded during the San Fernando earthquake, exhibited 30 to 40 percent longer period characteristics than predicted from the idealized model of the structure with a fixed base foundation constraint. The difference was attributed[1] to the soft soils and pile-supported foundations which interacted with the soil to produce a longer period system.

As a rough guide to whether soil-structure modeling is warranted, the following criteria have been suggested. Include soil-structure interaction:

1. If the shear wave velocity V_s at the foundation level is 1,000 ft/sec or less[7] and the soft soil extends to a depth equal to the smaller plan dimension of the building

2. If the soil index σ given by Veletsos and Meek[29] is less than 20,

$$\sigma = \frac{V_s}{fh}$$

where f = the structure's fixed base first natural frequency, and
h = the height to the centroid of inertia forces for the first mode of the fixed base structure.

The major effects of soil-structure interaction are to alter and lengthen some of the natural periods of the structure, and to modify the dynamic response of the structure, such as accelerations and forces. Whether this modified dynamic response is less than or greater than that of the fixed base structure will depend on several parameters.[29] The only truly reliable way of ascertaining the nature and magnitude of this modification is to analyze the structure using a model that includes the effects of the soil stiffness and energy dissipation characteristics, and compare this solution to the results of an analysis using a fixed base model.

Elastic Half-Space Method

There are essentially two commonly used approaches to modeling the stiffness, inertial, and energy dissipation characteristics of the soil. The first of these is referred to by several names, including the continuum approach, the compliance method, the impedance or spring-dashpot method, the elastic half-space method, or the lumped parameter method. This method utilizes equivalent lumped parameter springs and dashpots to model the distributed properties of the assumed uniform soil continuum (see Figure 1b for an example). Normally translational springs and dashpots and a rocking (rotational) spring and dashpot are utilized. These equivalent discrete stiffness and damping properties are derived from continuum mechanics utilizing the results of harmonically vibrating rigid circular foundations on an elastic half-space.[30] The resulting elastic half-space spring and dashpot coefficients are in general a function of the applied forcing frequency. However, the frequency dependence of these parameters is commonly neglected.

Recent developments have extended the elastic half-space method to include consideration of layered sites[31] and in an approximate manner, the effects of foundation embedment.[32] Also, the development of lumped parameter springs and dashpots from visco-elastic half-space theory rather than the elastic half-space has been suggested by Veletsos and Verbic.[33]

Finite Element Method

For complex soil layering (particularly nonhorizontal layering), for embedded foundations, for foundations of complex or irregular geometries, for flexible foundations, or where multiple structures are founded adjacent to each other, the continuum or elastic half-space method may not satisfactorily or reliably approximate the effects of the soil. In such cases, the second main approach to modeling the effects of soil, namely the finite element method, is utilized.

There are several possible ways in which the finite element method is applied to soil-structure interaction modeling. One approach is to construct a finite element model from which the equivalent static lumped parameter stiffness of the surrounding soil may be determined directly.

The second approach and the most common is to construct a single finite element model of the structure, foundation, and soil and input the free field ground motion at the boundaries of the soil model. The typical soil-structure interaction problem is in reality three-dimensional. Although use of a true three-dimensional finite element model for a structure and the underlying soil is theoretically feasible, such models are usually impractical or excessively expensive. The only major exception is when the structure is of such a nature as to permit modeling by use of axisymmetric elements. The prime example of such a structure is one that is cylindrical, such as nuclear containment vessel. With this important but unusual exception, most finite element models of structure plus soil are necessarily two-dimensional, and utilize plane-strain elements to approximate the soil continuum.

Typical two-dimensional nonlinear finite element programs that solve the dynamic problem in the frequency domain are LUSH[34] and FLUSH.[35] For axisymmetric structures modeled with finite elements consisting of shells and solids of revolution, the elastic time-domain solution may be solved on ASHSD-2[36] and the equivalent nonlinear frequency domain solution with ALUSH.[37] If the problem is small or the budget large enough, the three-dimensional soil-structure interaction problem may be handled with programs like SAP IV[12] or NASTRAN[13] in the time-domain with elastic models or with NONSAP[17] or ADINA[19] for inelastic models.

Foundation Modeling Assumptions

For conventional frame buildings on medium-to-firm soils, soil-structure interaction is normally neglected. For shear wall structures on soft soils, modeling of soil-structure interaction by the lumped parameter approach is becoming more common. Generally strain-dependent dynamic soil stiffness (shear modulus) and damping properties are established for the site soil profile through a combination of field exploration, laboratory testing, and computer analysis. The profile is modeled as a one-dimensional shear beam, and the design earthquake motion is used as input to the solution. The typical program used in this analyses is SHAKE.[38]

Equivalent elastic springs and dashpots are evaluated for modeling the soil stiffness and damping by elastic half-space methods. The dashpot damping property is usually the combination of material damping in the soil at earthquake strain levels along with radiation damping. The different damping in the soil and structure generally presents a problem in the linear elastic analyses, if modal damping is required as input to the computer program. There are approximate procedures for solving the problem.[7] However, work needs to be done to make variable material damping a more readily input parameter to computer programs.

The elastic half-space solution assumes a rigid foundation on the surface of an elastic media. For most spread footings, this is a fair approximation. For flexible mat foundations, the approximation is crude. Furthermore, for buildings with two or three basement levels, the footing can no longer be considered on the surface. In addition, basement walls must be designed for dynamic soil pressures under earthquakes. The only valid method for representing the wall, embedded footing, and soil stiffness is with a finite element model in this situation.

Another foundation system that has been very grossly modeled for earthquakes is the pile foundation. Little has been done to develop field and office methodologies for defining the lateral and axial deformation relationship for piles under earthquake when the surrounding soil stiffness is reduced due to the large seismic strains. Analytical methods for modeling the nonlinear soil stiffness and damping behavior along a pile have been recently proposed, and methods for modeling soil distorton along the free field boundary of the pile have been developed.[39] However, the field and laboratory test procedures to develop meaningful parameters for computer input have not been fully demonstrated.

SUMMARY OF MODELING UNCERTAINTIES

Some of the major areas of modeling uncertainty and general assumptions have been pointed out in this paper. There are many others which the reader should add to the list from his personal experience. The following is a summary of the major areas where further research and development would have the greatest impact on modeling of reinforced concrete buildings.

1. Diaphragm modeling--Methods should be developed for properly identifying the degrees of diaphragm flexibility for selection of the appropriate modeling approximations, such as semirigid, flexible, etc. Improved computer programs need to be developed which permit the minimum of added input data to model flexible and perforated diaphragms.

2. Section property--Refinements need to be made in the assumptions used to establish effective section properties in reinforced concrete members. The appropriate section is dependent on deflection or distortion in the member as the section cracks, but in addition there is uncertainty over what the effective section should be when modeling "T" beams, "L" shaped spandrals and intersecting walls.

3. Nonstructural members--Further definition and research is needed in the stiffening and damping which nonstructural elements may contribute to a structure in terms of period, mode shape, and force redistribution. Also, what is the form of nonstructural stiffness degradation? Is it abrupt, forcing the structural system to absorb large impact loadings or is it gradual?

4. Structural damping--Damping has a major effect on dynamic response and is one of the more significant areas of modeling uncertainty. We know certain concrete materials have higher damping characteristics than others. Have we pushed the state-of-the-art in this area to the limits of current technology? Are there concretes which could deliver damping values as high as 15 or 20 percent, at strain levels approaching the yield point in steel reinforcement, without losing the present qualities of strength and ductility?

   Are there ways of developing better estimates of damping in the higher modes of vibration? San Fernando earthquake data from the instrumented buildings showed a trend toward reduced damping in the higher modes.[1]

5. Soil-structure interaction--Soil stiffness, mass, and damping are all vital elements in the earthquake response of buildings. Current building codes recognize site-structure resonance as a design consideration. Proposed building codes suggest inclusion of soil stiffness in structural period calculations. Further work is needed to establish the overturning resistance contributed by soil inertia around footings and piles, the potential benefits

of hysteretic damping in the soil at large deformation, the softening of soil stiffness properties under foundations at large earthquake strains, and improved testing procedures for defining the nonlinear characteristics of soil required for meaningful modeling. There are many fruitful areas of research in soil-structure interaction which could significantly improve the accuracy of analytical models and improve our understanding of building-foundation behavior.

6. Mass properties--Building mass and its distribution are commonly thought to be the best defined quantity of all the modeling variables. This may be true for the majority of structures. However, the light high-rise structures currently under construction may be very sensitive to live load mass redistribution. Torsional effects under variable mass distribution need to be examined and guidelines developed for engineers performing dynamic analysis. No accidental mass eccentricity is considered in most dynamic solutions.

7. Inelastic properties--There are still major areas of research required in defining the nonlinear stiffness and damping characteristics of structural and nonstructural elements. Once the force-deflection relationships are established, research should be devoted to the modeling of failure mechanisms and damage states. As engineers we will be required in the future to provide better estimates of what the economic trade-off is between initial construction costs and earthquake repair costs. Such cost-benefit analyses are currently being performed with limited data and approximate models.

In summary, there are still many areas of research and development that could improve the modeling techniques and refine the idealizing assumptions used by the practicing structural engineer. We have progressed rapidly over the past 15 years in the art of structural modeling. Hopefully, research and technology will permit similar advances over the next 15 years.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED
CONCRETE BUILDING CONSTRUCTION (ERCBC)
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MODELING OF REINFORCED CONCRETE BUILDINGS

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I. INTRODUCTION

I. A. General Considerations

In the present report, modeling will be considered to be the process of representing prototype reinforced concrete (RC) building behavior with mathematical equations.

Design engineers, building officials, and structural analysts have different motivations for modeling. The design engineer uses modeling to satisfy himself that the selected configuration satisfies the load and serviceability requirements. The building official wants modeling to prove that the proposed structure satisfies code requirements. The analyst is more interested in the veracity of the model.

The three engineering groups want the model to prove that there's no chance for a failure of the prototype, but that's an unattainable goal. The most they can hope is that the model will reveal a defect in the structure when it is subjected to a variety of gravity load, wind load, and ground motion inputs. Presumably, the inputs will exceed the code requirements.

Why is modeling so popular? There are several reasons. It's fast, inexpensive, and can represent a prototype before it's constructed.

The principal difficulty with modeling is its veracity.

I. B. Individual Judgment in Modeling

The understanding of, knowledge of, and experience with response of buildings is a basic ingredient for modeling. In other words, the numerical results from modeling are simply a confirmation of what experience tells will occur.

Several corollaries illustrating the need for judgment in modeling are now offered. They are:

1) If the analyst isn't aware of the phenomenon to be represented, he probably won't find it from his modeling.

2) A mode of failure is easily overlooked and analytically excluded by the analyst who isn't aware of it.

3) The analyst must decide which phenomena to represent on the basis of their relative importance.

Judgment comes from exposure to building behavioral phenomena. The number of phenomena to be represented seems endless. A few are offered at this juncture, for illustration of the exposure that is required. Some of the offered phenomena are associated with failure or collapse, while others are related to performance of the structure while it's operating in the working range. The offered phenomena have one thing in common - they're not usually represented in structural analysis models.

Consider eight modes of behavior which are illustrated in Figs. 1 to 8. They are:

1. Various forms of buckling (Fig. 1) in reinforced concrete buildings range from buckling of longitudinal reinforcement to buckling of elements such as column and walls to buckling of complete building systems. Buckling effects are represented in RC design, using the magnifier concept.

2. Dynamic impact between adjacent buildings (Fig. 2) occurs during earthquakes. The effects of the impact on response of the adjacent building systems aren't well understood, but it's clear that local damages occur because of the impact. The problem is avoided through design; the code requires a separation between buildings which will supposedly keep the buildings apart during earthquake response.

3. Diaphragm deformations (Fig. 3) occur when there are in-plane normal and shear force resultants in the floor system. The in-plane forces are generated when the floor acts in the load path for the lateral loads applied to the building. Diaphragm forces are treated in design by providing appropriate reinforcement.

4. The vertical deflection and rotation of footings will occur under working loads (Fig. 4). Elastic responses will accompany the working loads. Under severe load conditions, footings may fail according to one of the following modes: a) permanent deformation of the soil surrounding the footing, b) lift-off of the footing away from the soil, and c) shear, anchorage, of flexural failure of the RC footing. The failure of the footings has disastrous consequences for the building. Footings are designed to operate in the working load range where their behavior is understood. At ultimate, the behavior is too complicated to predict.

5. The zone of slab adjacent to columns is subjected to moment, shear, and torque as the building responds laterally (Fig. 5). In design, it's important to recognize that punching shear failures occur when there are large drifts. Reversed punching shear may cause vertical collapse of the slab system. When modeling, the contribution of the slab system to lateral stiffness and resistance of the building is usually neglected.

6. Shear failure and hinging (Fig. 6) in beams and columns often occur when a building is subjected to strong lateral loads. The shear failure of columns is often fatal for the building. Seismic codes endeavor to prevent shear failures by providing lateral reinforcement to prevent the failure. Representation of the failure is not included in most models.

7. Out-of-plane vibration of the slab and wall systems with natural periods and frequencies which fall in the same range as lateral vibration modes of short buildings (Fig. 7). This phenomenon usually doesn't have significant design consequences but may complicate eigenvalue analysis procedures.

8. Overturning forces are a consequence of system moment due to lateral loads on the building (Fig. 8). Column and wall axial forces and moments due to lateral loads may be called overturning forces. These element forces cause axial, hinging, and buckling failures in column and wall elements. The effect of the failures is not considered in analysis models, but designs account for the overturning force effects.

The above cases show that many things must be considered when modeling is attempted, and why understanding of prototype behavior is so essential for modeling.

I. C. Objective

Current practices in specific aspects of modeling and analysis for RC buildings are discussed. The present report is limited to discussion of selection of coordinate systems and stiffness models. These are topics which are not usually discussed in research articles and textbooks concerned with structural analysis and structural dynamics. The numerical analysis aspects and solution algorithms with details on eigenvalue methods, nonlinear incremental techniques, and simultaneous equation solving are not discussed.

I. D. Definitions

Throughout the present report the terms elastic, inelastic, linear, nonlinear, limit state, incremental and dynamic will be used when referring to modeling and analysis. The following concepts will be attached to each of these modeling terms:

1. Elastic - material or element which exhibits a linear relationship between force and deformation.

2. Inelastic - material or element which exhibits a nonlinear relationship between force and deformation.

3. Linear - material, element, or building system which exhibits a continuous straight line relationship between force and deflection, inertia force and acceleration, and damping force and velocity.

4. Nonlinear - material, element, or building system which exhibits a discontinuous and/or curved relationship between force and deflection, inertia force and acceleration, and damping force and velocity.

5. Limit state - internal forces in and applied forces on the structure when a collapse mechanism is formed in the structural system.

6. Incremental - time or load response history of the structural system divided into small parts.

7. Dynamic - time history or maximum response of the building system due to loads characterized by a time history, e.g. earthquake loads.

II. CONNECTION OF ANALYSIS WITH DESIGN PHILOSOPHY

When the ACI decided to go to ultimate strength design in 1963 (1)[1], it offered formulas for computing the ultimate resistance or capacity of RC elements. The code did not offer analysis procedures for predicting the response of the structure in its ultimate state. Instead, the ACI code recommended that the design loads should be determined by the theory of elastic frames (linear analysis) even though limit analysis methods (2-4) developed for steel and RC structure analysis could have been adopted. In the 1971 code (5), the elastic frame analysis model continued to be recommended.

Of course the load and ∅ factors are used to increase the working loads up to an ultimate design load level, but still the linear analysis approach is not consistent with the RC element resistance evaluation procedure.

Why not?

The inconsistency can be demonstrated in several ways. One way is to consider first yield versus mechanism formation in elasto-plastic incremental analysis. Elastic analysis has the capability of telling when initial element yield will occur. This implies that only one element in the structure has reached its capacity. Usually, the load can be increased significantly before enough elements have yielded to form a failure mechanism in the structure. The elastic analysis does not represent the behavior in the range between first yield and mechanism formation. The factored version of the elastic result is usually an untrue representation of the actual force distribution when the building reaches its failure mechanism capacity.

Why then are these seemingly untrue forces found from linear analysis sanctioned by the ACI Code?

The elastic frame analysis requires less data and fewer computational steps. Therefore, the cost is low, more computer programs are available, and the difficulty in using the programs is not so great. For these reasons, most practicing engineers favor elastic analysis models.

1 Numbers in parentheses refer to corresponding items in the Bibliography.

Further, it has not been clearly demonstrated that a better performing design can be found using nonlinear incremental or limit state models. In other words, it is feasible to design a building with good performance characteristics using elastic analysis methods, so why use nonlinear analysis methods? Furthermore, many uncertain factors related to nonlinear behavior influence the performance of prototypes. It may not be accurate to consider just a few of these factors and it is computationally prohibitive to consider many of the factors.

These are the powerful reasons which are used to justify the continued use of a procedure which matches ultimate element capacities with factored working resultants found from elastic frame analysis.

Still there's an uneasy feeling when using elastic frame analysis for design. The ultimate internal force distribution isn't known. This lack of information provides justification for use of nonlinear analysis methods in representing ultimate behavior.

It is known that structures operate at the ultimate levels because of prototype failures in earthquakes. Then, a principal argument for using nonlinear incremental analysis is based on the need to represent the building undergoing the ultimate response that is likely to occur during a strong earthquake.

There is nothing in the immediate future which indicates that elastic methods will be replaced by nonlinear methods as the principal analysis procedure to be used for RC design. A significant decrease in computing cost would make the change more feasible.

Modeling methods are strongly influenced by the choice between linear and nonlinear approaches. In nonlinear modeling, a great deal of information regarding material and element properties must be supplied. Also, more displacement coordinate information is needed in nonlinear modeling because the patterns of deformation are more localized and complex, e.g. representation of the hinging zone and its growth along an element due to strain hardening requires a complicated set of displacement coordinate functions. All of the additional information requires several magnitude increases in computer storage and computer time for handling.

III. COORDINATE IDEALIZATIONS

It might be better to talk first about material and element properties, but the structural analyst is also taught to think of the building as a whole entity. Traditionally, he is educated to select a set of displacement/force coordinates which represent the building. Sometimes, he selects the finite element models to be used and then selects the coordinate system. In another approach he conceives the coordinate system first, e.g. rigid diaphragm three-dimensional idealization. Then he selects the elements to be used within the coordinate framework.

If a complete job is done, the analyst must consider coordinates and finite elements at the same time. He must have good judgment to select the correct elements and coordinates.

It's hard to write about both topics at the same time. In the present report, coordinate systems are discussed first, but the tie-in to finite element considerations is made when necessary.

The purpose of choosing a set of displacement/force coordinates for the building is to represent the motion and force behavior of the complete structure. Here is where knowledge of prototype behavior is essential, because the coordinate system must be selected to represent the patterns of displacement/force which are important in the structure.

III. A. Coordinate Representations for Modeling of Stiffness with Elastic Behavior

The force-deformation characteristics or stiffness must be accurately represented by the choice of force and displacement coordinates. Since buildings are made up from beam, column, wall, floor, and foundation elements, the logical choice for displacement coordinates are those which represent force-deformation characteristics in all of the building elements.

III. A.1. Two-Dimensional Coordinate Idealizations -

(Fig. 9) of building stiffness are used when the prototype has no twisting. Separate two-dimensional analyses are conducted in each of the two planes parallel to moment frames and walls of the building. The planes may be called principal planes of building motion (Fig. 10). Components of load parallel to each of the principal planes are used in the analyses. The results of the two analyses are combined by vector addition to obtain the final biaxial result.

It is common for each node in a two-dimensional idealization to have three displacement coordinates. Correspondingly, every beam and column element will have three displacement coordinates at both of its ends. These coordinates will completely represent the forces and deformations within the elements provided there are no distributed forces or inelastic behavior between the ends of the element.

For a two-dimensional analysis in one of the principal directions, all of the frames resisting load in that direction are included in the analysis (Fig. 11). The parallel frames which are coupled by diaphragms in the prototype are linked by two-force elements representing the diaphragms. The number of frames included in the two-dimensional analysis can be reduced if there is symmetry and/or repetition of properties between parallel frames and diaphragm deformations are neglected.

If diaphragm deformations are neglected, it is possible to reduce the number of displacement coordinates in two-dimensional idealizations of building stiffness. When the floors are considered to be rigid against in-plane deformations, then the horizontal motion at all of the nodes on a single floor can be represented by one displacement coordinate (Fig. 12).

When one horizontal displacement coordinate is used to represent motion of the floor, then axial and shear forces within the diaphragm cannot be recovered directly by multiplying floor element stiffness by floor element displacement. However, it is possible to recover the diaphragm forces from equilibrium. The applied floor lateral load and lateral load carried by each frame or wall is known from analysis. Using this information, the diaphragm shear forces are found by writing an equilibrium relation for each frame or wall and adjacent diaphragms.

In lieu of physical assumptions, matrix methods can be used to reduce the number of required displacement/force coordinates. One technique for coordinate reduction uses the equilibrium equations. In static studies the equations have the form:

$$\begin{bmatrix} K_{aa} & K_{ab} \\ K_{ba} & K_{bb} \end{bmatrix} \begin{Bmatrix} U_a \\ U_b \end{Bmatrix} = \begin{Bmatrix} F_a \\ 0 \end{Bmatrix} \qquad (1)$$

in which U_a, F_a = displacement and load coordinates where loads are applied; U_b = displacement coordinates where no loads are applied; K_{aa}, K_{ab}, K_{ba}, K_{bb} = corresponding stiffness arrays. Eq. (1) can be condensed to the form:

$$\begin{bmatrix} K_{aa} - K_{ab} \; K_{bb}^{-1} \; K_{ba} \end{bmatrix} \begin{Bmatrix} U_a \end{Bmatrix} = \begin{Bmatrix} F_a \end{Bmatrix} \qquad (2)$$

In linear structural dynamics, the similar form to Eg. 2 is (6)

$$\begin{bmatrix} M_{aa} \end{bmatrix} \begin{Bmatrix} \ddot{U}_a \end{Bmatrix} + \begin{bmatrix} C_{aa} \end{bmatrix} \begin{Bmatrix} \dot{U}_a \end{Bmatrix} + \begin{bmatrix} K_{aa} - K_{ab} \; K_{bb}^{-1} \; K_{ba} \end{bmatrix} \begin{Bmatrix} U_a \end{Bmatrix} = \begin{Bmatrix} F_a \end{Bmatrix} \quad (3)$$

in which $\ddot{U}_a$, $\dot{U}_a$ = acceleration, velocity coordinates which have mass, damping forces; M_{aa}, C_{aa} = corresponding mass, damping matrices; F_a = load vector. For nonlinear problems, the condensation is required in each increment.

The use of Eqs. 1-3 in a numerical analysis context is clear. The benefit of the exercise in two-dimensional rigid floor idealization is not so clear. Often a lumped mass associated with the horizontal translation coordinate is used. It seems logical to associate U_a coordinates with floor horizontal translation and U_b coordinates with vertical displacement and rotation at the nodes. The condensation of so many coordinates and the complexity of the required coordinate numbering scheme make the problem complicated.

The condensation technique is very useful when used in other circumstances. One valuable application is on finite elements with interior coordinates.

Prototype beam, column, wall, slab, and foundation elements have changing or nonprismatic geometric properties along their lengths. A typical example occurring in two-dimensional geometric idealizations for a beam and column subassembly is shown in Fig. 13.

There are two ways to go when choosing the coordinate representation for nonprismatic beams and columns. They are (Fig. 13): 1) represent the stiffnesses at the end of the nonprismatic element; the algebraic formulas for the stiffnesses are obtained by a hand calculation method such as moment area; and 2) represent the nonprismatic element as a collection of prismatic elements with nodes between each pair of elements. The former is more efficient when the coefficients are known. When using the latter, "2)" method, the interior coordinates are the "b" coordinates of Eq. 2, while the exterior coordinates are the "a" coordinates.

Shear and bearing wall coordinate idealizations which are compatible with two-dimensional building analyses have three principal forms (Fig. 14):1) in one, the wall is made up from vertically oriented beam elements; three coordinates are used at each element end; 2) in another a panel idealization is used; three coordinates are assigned both at the top and bottom of the panel elements; 3) the most elaborate models the wall with planar finite elements; the number and type or coordinates depends on the type of finite element that is used.

Foundation coordinate idealizations (Fig. 15) compatible with two-dimensional building analyses either model a stiffness property at grade or represent the soil by a planar finite element idealization. The grade stiffness model uses a coordinate system that is compatible with the two-dimensional structure idealization. The finite element model uses a coordinate system that is compatible with the finite elements.

Behavior of floor systems under vertical load can be treated independently from lateral analysis. When considered as separate, it is a two-dimensional problem. If a plate-bending and beam finite element analysis model is used, then each node has one vertical translation coordinate and two horizontal rotation coordinates (Fig. 16). In-plane deformations of the slab are not considered in the two-dimensional characterization.

III. A. 2. - <u>Three-Dimensional Coordinate Idealizations</u>

Twisting or torsion response occurs when the center of mass and center of stiffness do not coincide (Fig. 17). This can occur because of building shape, arrangement of frames and walls, and distribution of mass. A three-dimensional idealization is necessary when twisting occurs because the motion of the building cannot be represented by superposition of motion in two orthogonal planes.

It is common for each node in a three-dimensional idealization to have six displacement coordinates. Correspondingly, every beam and column element will have six displacement coordinates at both of its ends. These coordinates will completely represent the forces and deformations within the elements, provided there are no distributed forces or inelastic behavior between the ends of the element.

If the floor and roof are idealized with shell finite elements, then slab and diaphragm behavior are simultaneously represented. These elements also have six degrees of freedom at each node.

A rectangular building (Fig. 17) with symmetry and coincidences of center of stiffness and center of mass has torsional modes of vibration. The torsional modes are not excited by translational ground motion because of the coincidence property of the building. A vertical rotation component of ground motion is the only component that will excite twisting in a building which has the coincidence property.

The importance of the vertical rotation ground motion component is not well established and is neglected in usual analyses. The torsional modes are automatically computed in three-dimensional analyses, but their participation factors are zero. Since there is no torsional mode participation, a two-dimensional analysis will provide the same result.

Setbacks cause twisting of building unless they are symmetric (Fig. 18).

Even in the symmetric setback case, three-dimensional idealizations are needed because the adjacent frames in the principal directions will have different magnitudes of displacement under uniform lateral load.

Diaphragm or in-plane deformations in floors occur because of: 1) transfer of floor inertial loads to frames and walls; 2) compatibility forces which come from maintaining of near equal horizontal displacement of walls and frames at each floor level; 3) transfer of shear from one vertical wall, bracing system, or frame element through the floor to an adjacent shear resisting vertically alligned element; e.g. the staggered truss system. A three-dimensional idealization is needed for representing diaphragm deformations even when planar motion in the principal planes of building motion occurs. The adjacent frames lag behind one another due to shear and bending deformations in the plane of the floor diaphragm.

If diaphragm deformations are neglected in three-dimensional analysis, it is possible to reduce the number of horizontal displacement coordinates to three (Fig. 19). When the floors are considered to be rigid against in-plane deformations, then the two horizontal translation and one vertical rotation coordinate at each of the nodes on a common floor can be represented by a single floor node triplet of coordinates consisting of two horizontal components and one vertical rotation component.

Diaphragm forces cannot be evaluated when the rigid diaphragm idealization is used. The forces transferred into frames and walls at each floor level can be evaluated, but the problem of determining the distribution of diaphragm stress resultants due to these forces cannot be solved accurately by equilibrium methods.

The condensation techniques used in Eqs. (1-3) can be used in three-dimensional analyses. The techniques have the same strengths and weaknesses as were discussed for two-dimensional models.

For nonprismatic beam and column elements, the three-dimensional coordinate idealizations either use the equivalent end stiffness or interior node coordinate system. The technique was discussed for two-dimensional idealizations.

Wall coordinate idealizations may be generalized to three dimensions. The coordinate idealizations for the three element types are: 1) beam element with out-of-plane bending represented by 12 coordinates; 2) panel element - out-of-plane stiffness is not represented so there is no difference between the two- and three-dimensional idealizations; 3) a flat shell finite element idealization is the same as is used for the three-dimensional floor idealization; both in-plane and out-of-plane deformations are represented; 6 coordinates per node are used.

Foundation coordinate idealizations in three-dimensions like their two-dimensional counterparts are composed of two basic types: 1) 6 stiffness coordinates expressed at grade; and 2) three-dimensional finite element model.

In buildings the number of required displacement coordinates goes up rapidly in three-dimensional idealizations which use six degrees of freedom at every node. Compared with two-dimensional idealizations, twice as many coordinates are needed at each node. Also, many more nodes are needed because of the third dimension of the building that is modeled. Even when the rigid diaphragm idealization is used, three coordinates per node remain to be represented as system coordinates. Therefore, the rigid diaphragm reduces the required coordinates by 50% when compared with the full three-dimensional idealization.

III.A.3. Coordinate Idealizations Used in Programs for Representing Elastic Behavior

Three types of coordinate idealizations appearing in currently used three-dimensional programs can be identified. They are: 1) Type U) - unrestricted coordinate idealizations (7); Type R) - coordinate systems restricted to building idealizations (8); Type S) - coordinate systems special to specific individual buildings (9). The list of referenced programs is not purported to be complete. Instead, the programs referenced are intended to indicate how currently used programs fit into Type U), R), and S) coordinate idealizations.

When Type U) programs are used, an elaborate coordinate idealization is possible. In these programs beam, plate, shell, and solid finite elements are assembled to represent the complex geometry and motion of the structure. The coordinate idealization is automatically settled by the choice of the finite elements. An experienced analyst will choose a sufficient number of elements to adequately represent the structure. An inexperienced analyst either will choose too few or too many elements. If too few are chosen, then some important pattern of behavior will be overlooked, e.g. see Fig. 20. If too many elements are used, then several things may go wrong (Fig. 21): 1) computer and data preparation cost will be high; 2) the math model will give too flexible a representation; 3) the fundamental mode shapes will represent unwanted patterns of displacement; 4) numerical problems will occur in the solution of equations; 5) too much information will be obtained.

Type R) programs are written specifically to represent building systems (8). These programs are ideally suited to earthquake modeling of frame and box systems. The programs contain finite elements which are intended to represent accurately the behavior of the structural elements which are normally found in buildings. Displacement coordinates are restricted (Fig. 19) and chosen to represent only the relevant displacement patterns in the building. In most instances, this leads to a reduction in coordinates, elements, and computer costs when compared with Type U) programs. The disadvantage of the Type R) programs is that important modes of behavior could be overlooked, e.g. the increased flexibility due to an opening in a wall might be neglected. Here the maturity of the analyst is tested in a different way. When using Type R) programs, the analyst must decide if the finite elements included with the program have geometric and stiffness properties which adequately represent the elements of the prototype.

With Type S) programs written for special buildings, the programmer and analyst must work closely together or be the same person. The motivation for writing Type S) idealizations is to reduce the number of coordinates and elements to a small number.

III. B. Coordinate Representation for Modeling of Stiffness with Inelastic Behavior

The selection of displacement coordinates which are appropriate for representing inelastic behavior is more complicated than for representing elastic behavior. The inelastic force-deflections characteristics of the various structural elements are not understood except for the simplest beam configurations. Much more subassembly and full-scale experimental testing is needed before the inelastic model results can be verified.

In spite of the uncertainties, models have been developed for representing inelastic behavior of certain prototype elements, appropriate coordinate systems have been chosen, and incremental analyses performed.

In most instances the coordinate systems have been patterned after those used for representing elastic behavior. That's a logical step, but important patterns of deformation may be overlooked with these coordinate systems.

III. B. 1. Two-Dimensional Coordinate Idealizations of building stiffness are used for representing building response when the motion is parallel to one of the principal planes of motion. It is not correct to combine the inelastic results for two directions by vector addition to obtain the final element forces and displacements. There is a strong interaction of the two components, i.e. the stiffness in one direction is affected by the load and deformation in the other direction. The sequence of component application or loading history also has a large effect on the stiffness. As with linear modeling, two-dimensional idealizations are not valid when twisting occurs.

Two-dimensional modeling follows the approach used for linear studies. The representation of parallel frame systems is carried out, using the two-force element link beams discussed for linear models. Three displacement coordinates are used at each joint of the frames and walls that are represented.

For inelastic modeling, it is computationally expedient to eliminate coordinates which represent insignificant deformations. The coordinate reduction method based on rigid floor diaphragms can be used effectively in this regard.

The condensation technique for reducing numbers of displacement coordinates also can be used but with some restriction. The method given by Eq. 2 can be used in incremental procedures but the condensation must be performed in every increment. The K_{aa} and $K_{ab} K_{bb}^{-1} K_{ba}$ terms will change between increments because of inelastic behavior. If the K_{aa} and $K_{ab} K_{bb}^{-1} K_{ba}$ matrices remain constant during the sequence of analysis, then Eq. 2 needs to be used only once. The former condition will occur when that portion of the structure associated with coordinates "a" and "b" remains elastic during the simulation.

Beams and columns undergoing inelastic responses have behavioral characteristics which resemble those of geometrically nonprismatic elastic elements. For example, hinging zones which occur where the ratio of applied moment/resistive moment is approximately equal to unity, can be considered as regions of low flexural stiffness.

The hinging zones (Fig. 22) usually have lengths less than twice the beam depth. Even in beams with prismatic capacities, the hinging zones will occur. This is true because moment diagrams will vary along the length.

If the beams don't have prismatic capacity, then the hinges will move away from the usual locations at the ends and at the midspan. It is possible to represent hinging zones and nonprismatic segments in beams, but some foresight is required.

Inelastic behavior is modeled in these elements, using many different coordinate idealization approaches. Two methods (Fig. 23) receiving considerable use for beam elements are: 1) the combined elastic and hinged plastic end element (10, 11, 17) and the interior coordinate element (12). The "2)" method is more powerful than the "1)" method, but it requires more coordinates.

The "1)" method can be used effectively if the moment-rotation stiffness of the two coupled beam end coordinates can be established for complicated loading paths.

The "2)" method which uses interior coordinates is simpler to formulate because it works with segments or zones of prismatic flexural-stiffness. It chooses an EI for each segment by tracing the average moment curvature history of the segment. A sufficient number of segments and interior coordinates must be chosen to represent all zones where the capacity and stiffness are constant and where hinging will occur, (Fig. 23). The number of required coordinates goes up rapidly, since the additional ones necessary to describe the inelastic behavior are required for all of the beam and column elements used in the system. This is one reason why inelastic modeling is costly. It's really quite wasteful unless inelastic behavior occurs at many locations in the structure.

Shear and bearing wall coordinate idealizations appropriate for representing two-dimensional inelastic behavior are patterned after elastic idealizations. Beam, panel, and finite element coordinate systems have been tried.

Beam and panel models assume a gross element inelastic behavior which obey complicated force-deflection rules expressed at nodes which are positioned at floor levels throughout the wall height. These are bending and shear models with three coordinates at each element end. They are simple models representing complex two-dimensional behavior in the wall. The complexity of wall behavior is discussed in Ref. 13. The models trade off completeness of representation for computing efficiency. If the rules were complete and accurate for representing participation of the wall in building performance, then it would be safe to use them.

At this juncture, there isn't sufficient data from appropriate tests to get a complete fit with these models. Current testing is helping to give badly needed information, but experimental evidence is still not complete because there are so many different prototype configurations and loading conditions.

Finite element models (25) try to describe behavior at many locations throughout the wall. The coordinates and mesh must be chosen to represent all of the complex phenomena which occur in a wall. Among these phenomena are: cracking, reinforcement slip, crushing, buckling of longitudinal bars and shell spalling in the boundary frame, and slippage at floor construction joints. It's possible to create the coordinate system that will describe all of these phenomena, but the nonlinear force-deflection rules are not easily found. In the end, some sort of system identification method is used to evaluate the parameters controlling the rules so that the gross behavior as described by the finite element model agrees with the gross behavior found from experiments.

The computing cost for the nonlinear finite element exercise is high and the end product is of questionable value. The beam and panel models will be more cost effective and more reliable after the force-deflection rules are known.

Foundation systems can be represented with two-dimensional coordinate idealizations resembling the ones used for linear systems. To use the "at grade" model, then inelastic force-deflection rules for the footing and pile systems must be known. The alternate idealizations by finite elements are plagued with the same difficulties as those encountered in modeling of walls.

Inelastic modeling of slab systems (14) can be treated with the same coordinates as were used for linear systems. The coordinate and mesh configurations are selected to represent the regions of hinging. The regions resemble strips of width comparable with the slab depth and running along yield lines.

III. B. 2. Three-Dimensional Coordinate Inelastic Idealizations are used for inelastic studies when there are two components of ground motion to be represented or when twisting occurs.

Beam and column elements are idealized for three-dimensional behavior using two types of coordinate idealizations. Both idealizations use interior coordinates.

In one (15), the stress-strain history is retained for many locations in the element. In each increment the tangent modulus of the stress-strain law is integrated over the volume of the element to obtain the stiffness at the ends of beam and column element.

The second method (12) uses biaxial moment curvature yield rules for different segments along the length. The element end stiffness is found from condensation.

Three-dimensional coordinate idealizations for diaphragm, shear wall, and foundation inelastic behavior can be formulated. It is not feasible to attempt these formulations at this juncture. There will be may uncertainties in the idealization because of complexity of these important parts in building systems.

III. B. 3. Coordinate Idealizations Used in Programs for Representing Inelastic Behavior

Type U) programs include NONSAP (16) which is a generalized version of SAP (7), but with classic inelastic effects represented.

Type R) programs include two- (11, 16, 17) and three-dimensional (12), versions.

Type S) programs (9, 15) are especially useful for reducing computer costs.

The principal drawbacks in three-dimensional inelastic programs are: 1) the excessive computer cost for their use, and 2) the doubt that a reliable result is obtained unless there is experimental verification.

III. C. Mass Coordinates

It is theoretically sound to derive consistent mass matrices (19). When this is done, there is a matching of nonzero terms in the system mass and stiffness matrices, and all diagonal terms of the mass matrix are positive. For building studies, it's not worth the effort, because approximately equal building responses are obtained when the mass is represented with the lumped mass approximation.

In the lumped mass idealization, positive mass values are assigned to certain diagonal elements of the system mass matrix. The remaining matrix elements are zero. The matrix row and column numbers of the nonzero elements correspond to the translational or rotational components which have mass associated with them. The translational mass is found by computing the tributary mass for the coordinate. The rotational mass is found by taking the rotational mass moment of inertia about the corrdinate location.

In most building studies, the mass values remain constant. Nonlinearities due to changing mass will not be discussed in the present report.

III. C. 1. Two-Dimensional Mass Coordinate Idealizations

The mass coordinates are chosen to be compatible with the stiffness coordinates.

In the two-dimensional idealization, the mass is lumped with the horizontal acceleration component at each floor node (Fig. 11). The magnitude of the lumped mass is found by computing the tributary slab, beam, wall, column, and nonstructural dead load for the node.

When the diaphragm deformations are neglected, then the mass of the floor is assigned to the horizontal acceleration coordinate (Fig. 12).

The condensation technique in its simplest form is given for dynamics application in Eq. 3. In this case there is no mass assigned to the "b" coordinates. For example, this would occur in the rigid diaphragm assumption where the only mass is assigned to the horizontal floor translation coordinates, i.e. "a" would be floor translations, while "b" would be all others.

When performing inelastic incremental procedures, the condensation technique can be useful for saving core but considerable computation is needed. Step-by-step incremental equilibrium balance solutions used in inelastic studies require that equilibrium balance is achieved at all coordinates including "a" and "b". In contrast with linear solutions, the mass must be represented at the "b" coordinates because otherwise it will be impossible to represent inertia force properly in the equilibrium balance equation. A diverging solution is obtained when the mass from the "b" coordinates is simply lumped at the "a" coordinates.

In inelastic dynamic studies where interior coordinates are used for beams, columns, floors, shear walls, and foundations, the equilibrium balance must be at these coordinates along with the system coordinates which tie the building together (Fig. 24).

Vertical acceleration studies of floor systems can be performed with two-dimensional idealizations. Lumped mass and stiffness coordinates must be assigned to nodes along the beam elements and at interior nodes of slabs. If that is done, the vibration of floor and beam systems will be represented together with up and down building modes due to lengthening and shortening of the columns.

III. C. 2. Three-Dimensional Mass Coordinate Idealizations

The system mass is lumped with the three translation coordinates at each node. It can be automated in the program and be assembled from element mass matrices. This procedure is valid for beam, plate, and shell element idealizations.

When considering three-dimensional building analyses where the diaphragm deformations are neglected, the mass of the floor is lumped for use with the two translation and vertical rotation coordinates. The mass matrix evaluated at the center of mass (COM) will have the form

$$\begin{bmatrix} M_x & & \\ & M_y & \\ & & M_{\theta z} \end{bmatrix}_{COM} \tag{4}$$

in which M_x, M_y, $M_{\theta z}$ = translational x, y and rotational θZ masses. The rigid diaphragm assumption admits a transformation between COM and floor coordinate point P (Fig. 19). It is

$$\begin{Bmatrix} U_x \\ U_y \\ \theta_z \end{Bmatrix}_{COM} = \begin{bmatrix} 1 & 0 & -\bar{y} \\ 0 & 1 & \bar{x} \\ 0 & 0 & 1 \end{bmatrix} \begin{Bmatrix} U_x \\ U_y \\ \theta_z \end{Bmatrix}_P \tag{5}$$

in which U_x, U_y, θ_z = translation x, y and rotation Z displacements; $\bar{x}$, $\bar{y}$ = x and y distances from the COM to P. Transformation (5) is used to express the mass matrix (4) at a coordinate location that is convenient for use with the stiffness coordinates.

III. D. Damping

Damping is not usually regarded as a quantity which is directly related to a physically identifyable nodal velocity coordinate. Instead, the damping energy dissipation is related for computational convenience to the system stiffness and mass matrices. In linear studies it's often related to the mode shapes (20). The generalized damping force for each mode is expressed as a set percentage of critical damping.

For inelastic studies where it's desired to retain the linear mode proportional damping concept, a damping matrix can be constructed by "working backwards" from the generalized damping matrix. Let the damping matrix, C, be given by

$$\left[C \right] = \left[\Phi \right] \left[2\lambda_i \omega_i \right] \left[\Phi \right]^T \qquad (6)$$

in which Φ = matrix of system eigenvectors; λ_i, ω_i = percent of critical damping, natural frequency of mode i.

The C matrix found in Eq. 6 can be conveniently combined with stiffness and mass matrices in step-by-step incremental procedures (21) used for representing inelastic behavior.

III. E. P - Δ Coordinates

Compressive axial loads in elements tend to decrease their lateral stiffness. In buildings, columns and walls are most affected.

A classical stability failure never actually occurs in an RC prototype. Instead, the axial loads tend to magnify the moments and shears that occur in the elements. The magnification may cause failure of column or wall elements or failure of the system (Fig. 1).

The two principal approaches used to treat the problem are: 1) geometric stiffness matrix approach (22), and 2) P-Δ shear approach (23). Both methods do not require modification of the coordinate systems used for representing two- or three-dimensional elastic or inelastic behavior. Method 1) is more suited to automatic computation.

III. F. Coordinate Systems for Representing Loads

In usual seismic applications, the loads are due to inertia forces which come from

$$\left[M \right] \left\{ \ddot{U}_g \right\} \qquad (6)$$

in which M = system mass matrix; $\ddot{U}_g$ = vector of ground accelerograph data. The load coordinates coincide with the mass coordinates.

Gravity effects are represented as distributed loads on beam and slab elements. The element end coordinates used for representing elastic behavior can also serve for the fixed end forces. The fixed end forces are found from the principal of virtual displacements.

In the event of inelastic behavior, the gravity loads are conveniently represented, using the interior coordinate approach. Fixed end forces representing the distributed load on each of the segments are applied at the interior coordinates.

III. G. Coordinate Systems - Final Comments

The selection of the coordinate systems is a crucial step in the modeling process. Engineering judgment is required because every building idealization has unique features. Computer structural analyses programs are heavily used at this juncture, but the structural engineer is not displaced because of this. One major contribution he will continue to make is in the selection of the coordinates to be used in the computer idealization.

IV. SELECTION OF FINITE ELEMENTS

The beam, plate, and solid finite elements used to represent the beam, column, slab, wall, and foundation parts in the building are fundamental to the modeling process. The element stiffness forming procedures (21) are used to find numerical values of stiffness and load to be associated with the displacement/force coordinates used to describe the structure.

The elements must be selected to represent the patterns of displacement/force which are important in the structure. The judgment needed for selecting the elements comes from knowing the prototype behavior and what the various finite elements will represent. Also, experience is needed in selecting the geometric and material properties used as input for the element stiffness forming procedures.

IV. A. Elements for Modeling Elastic Behavior

Several important modeling considerations concerned with elastic element usage are:

1) Type of element to be used, i.e. beam, plate, diaphragm, and solid.

2) Material properties used for the element.

3) Geometric properties of the element, i.e. width, height, length, area, shear area, moment of inertia, etc.

4) Weight or mass of the element.

It's helpful to discuss the considerations according to the type of element.

IV. A. 1. Beam and Column Finite Elements

These elements are used in two- (Fig. 9) and three-dimensional (Figs. 18 & 19) idealizations. The major difference in the dimensionality is that the former represents element deformation in a plane, while the latter represents three-dimensional deformation including twist.

Young's modulus, E_c, is the material property needed for two-dimensional beam elements which represent bending deformations. If shearing deformations are represented then Poisson's ratio, ν , is needed also. For three-dimensional elements E_c and ν are both needed.

Geometric properties include cross section and length dimensions.

The cross section geometric properties to be input may be specified as literal width and height dimensions or beam theory related area and inertia properties. If the inertia quantities are input, then a greater range of cross sectional idealizations is possible, but engineering judgment is needed for computing the quantities.

The basic idealization specified in the ACI Code (5) is based on gross section. Some of the considerations necessary for choosing cross section dimensions are presented in Table 1. Please note that cracked sections are used for serviceability calculations.

The magnitude of seismic response found, using a linear structural dynamics model, is strongly influenced by the values of modal periods of the structure. The effect is strongest when time-history or actual earthquake spectra are used for inputs. The periods are strongly influenced by the stiffness. Moments and shears within the linear working range can cause cracking which significantly reduces stiffness. The reduction in stiffness may cause an increase or decrease in response, depending upon the properties of the earthquake. This phenomenon leaves the engineer puzzled. Should he use cracked section or gross section when performing elastic linear dynamic analyses?

A reasonable solution to the problem is to analyze the building for both cracked and uncracked properties. For design, the more conservative result can be used.

In recently advocated long column analyses for unbraced RC frames (23), the use of inelastic EI values is recommended unless information on inelastic EI values is not known. In the latter case, an iterative analysis using linear gross EI values in each iteration is recommended. The EI's are as follows: 1) columns - 0.8 times gross EI; 2) beams - 0.4 times gross EI. These reduced EI's are intended to represent softening that occurs because of P behavior.

The length geometric properties of the beam and column elements are usually determined automatically in the programs. Sometimes a foreshortened clearlength element is used to account for the increase in rigidity through the joint. The clearlength element is recommended for use in RC modeling, but it does give an overly stiff representation because of apparent joint flexibilities due to anchorage slip and shear deformations in the joint.

Two variations of clearlength elements are used: 1) the element stiffness is computed with the shorter clear length and added to the coordinate stiffness of the adjacent nodes, and 2) the stiffness computed for clearlength element is transformed to the adjacent node using an ecectricity transformation similar in form to Eg. (5). The latter idealization is more correct because it represents the true connection between rotation and translation at the nodes and the ends of the clearlength element.

In certain programs the mass and weight of the element is computed. The only input needed for this is the material density, since the volume of the element is computed internally. The element mass is then assembled into system mass coordinates. Problems often arise in this type of mass formulation because the volume of structure doesn't coincide with the volume of structural elements used in the idealization.

IV. A. 2. Plate and Shell Elements

The shell elements are used for wall and floor three-dimensional idealizations in which in-plane and bending deformations are to be represented.

If a two-dimensional slab design is to be performed, then plate elements representing transverse bending may be used. Certain plate elements also represent transverse shearing deformations. These should be used when the span/depth ratio is less than five.

Young's modulus, E_c, and Poisson's ratio, ν, are the input quantities for these elements when gross section properties are assumed. If cracked properties are assumed, then the input is not so simple. The cracking has a preferential direction and the element has orthotropic properties. Reinforcement also has an effect on the orthotropy. Reference 14 considers some of the problems which occur.

The geometric inputs for the elements are mostly settled in advance by the selection of node positions. The only geometric input quantity is element thickness.

Mass and weight of the elements are computed through density input quantities.

There are some interesting modeling problems connected with use of plate elements. The problems focus on selection of elements and coordinate systems for adequate representation.

A beam and slab system represented by beam and plate elements is an interesting case for study.

Due to assumptions in finite element theory, the beam and plate elements are only linked at the nodes. The theory says that beam and plate elements can deflect by different amounts between the nodes, but in the prototype the two move together. As more nodes and elements are added, convergence of displacements will occur.

Another difficulty that occurs is to decide how to choose the moment of inertia of the beams. There is T-beam action between the slab and the beam. If the beam and slab idealization dimensions are chosen on the basis of nominal beam and slab cross sectional outlines, then an incorrect stiffness idealization will be obtained. A portion of the slab must be counted as flange of the beam in the region where the top of the beam is in compression. Where the bottom is in compression, a rectangular cross section must be used. This means the solution must be known in advance in order to select the correct beam inertias.

One method for doing this is perform the analysis in several iterations. Each time, the inertia is to change until it matches with the sign of the moment. The width of the T-beam flange to be chosen has a practical approximate solution given in Chapter 13 of Ref. (5).

To be correct it is necessary to choose beam element lengths to coincide with zones of constant sign. The extent of the zone may change between iterations.

This means new coordinates and beam inertias must be chosen in each successive iteration. This approach is not feasible for the designer. It is best for him to estimate the zones of positive and negative moment and choose the inertias according to the estimate. He then runs the program and uses the resulting moments and shears.

The plate finite element portion of the beam-slab floor system idealization requires some consideration. Even though part of the slab is used for beam flange, the same region of the slab is idealized with a plate element. This is necessary so that the bending of the slab between the beams is correctly represented.

Another plate-element-related problem of considerable importance is the shear wall with openings. The particular objective of the shear wall idealization must be established. Two obvious objectives to be considered are: 1) to represent the contribution of the wall to bhe lateral resistance of the building, and 2) to evaluate the stresses in the wall around the openings.

When working on objective 1) it is desirable to use coarse finite element grids. In regions where openings occur, it is feasible to use pseudo elements with a reduced thickness. The thickness of the pseudo element is chosen so that the stiffness for a pattern of deformation is in agreement between the pseudo element and a fine grid of elements representing a portion of the wall which has the same size as the pseudo element. For example, the thickness of the pseudo element could be chosen so that shearing stiffness of the pseudo element and the fine grid of elements around the opening shown in Fig. 26 are in agreement.

IV. A. 3. Diaphragm or Panel Elements

These elements are specifically designed (8) for representing in-plane behavior of walls. Shear and flexure beam modes are used in the representation.

Material inputs are E_c and ν .

Geometric inputs are by node specification. Also, element thickness is an input.

Density may also be input for the mass computation.

IV. A. 4. Solid Elements are used for representing general three-dimensional solid behavior. The elements have been used in ground motion studies (24).

They are rarely used in RC studies because of high computer cost, extensive data preparation, and uncertainties in reliability of the results.

IV. B. Elements for Modeling Inelastic Behavior

The principal considerations for various types of finite elements modeling inelastic behavior were discussed in Section III. B. - Coordinate Representation for Modeling of Stiffness with Inelastic Behavior. The discussion was presented in that section because of the strong connection between elements and coordinate systems. Further discussion is needed, but is of too limited application for inclusion in the present report.

IV. C. Element Selection - Final Comments

The key aspect in element selection is to match the appropriate finite element with prototype element behavior. Familiarity with finite element force and deformation patterns and prototype behavior are essential for this exercise.

V. CONCLUSION

From the present paper on modeling, it is apparent that computer structural analysis is a basic ingredient of structural engineering design. However, engineering judgment is still needed, even though the computer approach has been adopted. In the context of the present paper on coordinate systems and elements judgment is needed for selection of models to represent accurately prototype behavior.

The dimensionality of the coordinate system used for representing the RC building is a basic decision which the engineer has to make. This is the first of many decisions he makes when modeling a building. Other decisions are concerned with types of elements, elastic vs, inelastic behavior, foundation, mass, damping, $P\Delta$ effects, and loads. Some information which is helpful for making the decisions has been supplied in the present paper.

A lengthy discussion on selection of coordinate systems and elements to be used in the displacement or stiffness structural analysis approach has been included. It is emphasized in the discussion that selection of coordinate systems and elements must be done simultaneously.

The increase in complexity of coordinate systems and elements caused by choosing inelastic instead of elastic models also has been discussed. Further, the veracity and feasibility of inelastic models has been discussed. It is the intent of the present paper to give a balanced view on desirability of inelastic models. Certainly, the strongest argument in favor of them is their intent to give an "exact" mathematical representation of the structure during its final stages of resistance. Still a stronger argment for their use would be proof that a safer building could be designed when they are used.

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TABLE 1

COLUMN AND BEAM CROSS SECTIONS

USED IN ELASTIC STRUCTURAL ANALYSIS

Structural Element Type	Dimensionality	Purpose of Model	Computation of:
Rectangular Beam or Column	2, 3	Strength Design for Lateral Load	I, A, A_s, J from overall width and depth (Gross Properties)
T-Beam in Floor or Pilaster in Wall	2, 3	Strength Design for Lateral Load	I for bending transverse to floor, wall (Fig. 25A) I for bending in the plane of floor, wall (Fig. 25B) A - average of areas found for transverse bending (Fig. 25A) As for deformation transverse to floor, wall (Fig. 25C) As for deformation in plane of floor, wall (Fig. 25D) J-use T section found in Fig. 25A (Gross Properties)
Rectangular or T-Beam	2,3	Vertical Load Serviceability Analysis	Use cracked section properties for I when cracking moment, M_{cr}(5), is exceeded
Slab System	2	Equivalent Frame Analysis for Ultimate Strength Design	Use I and A values from gross cross sections specified in Chapter 13 of Ref. (5)

LIST OF FIGURES

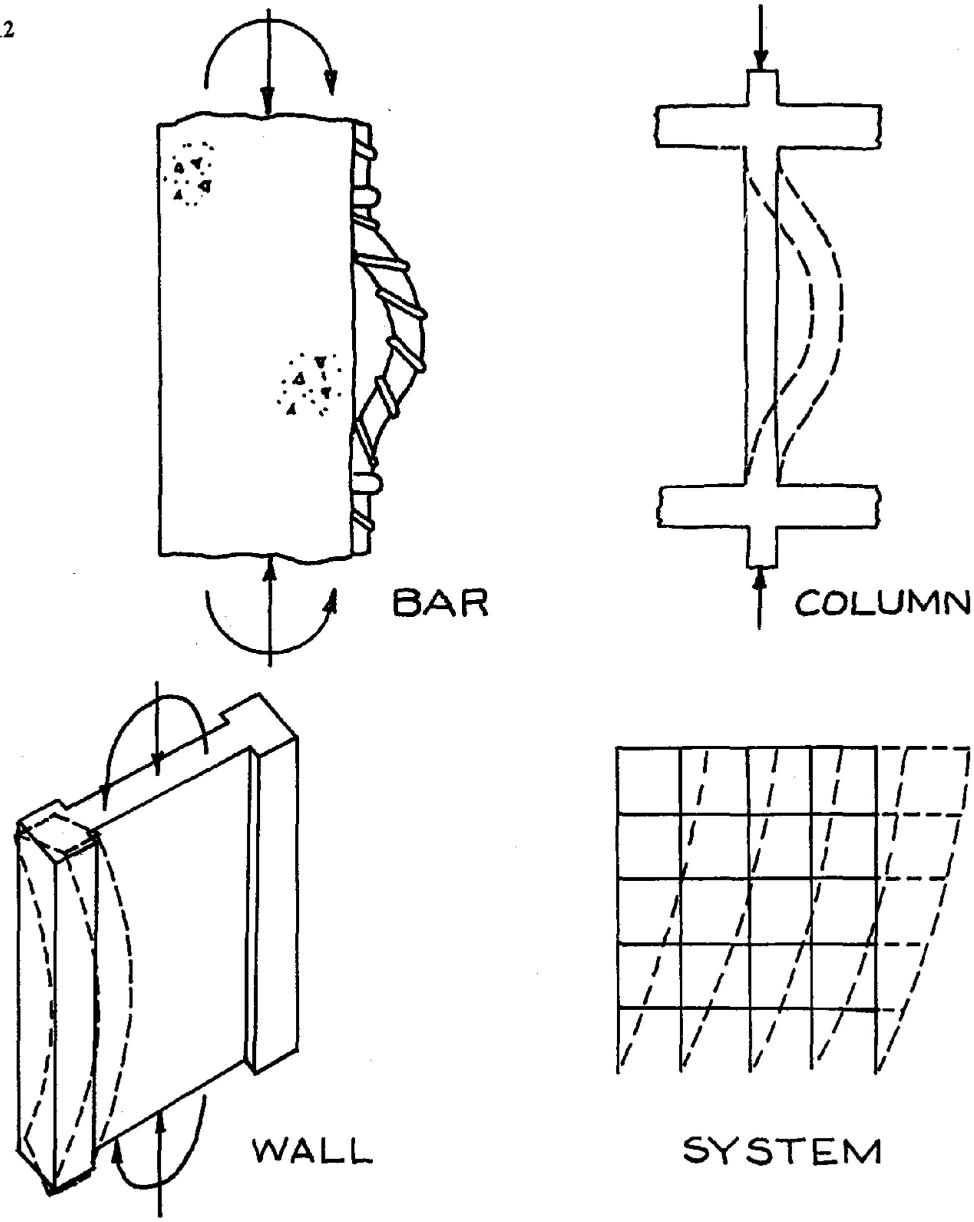


FIG. 1 - VARIOUS FORMS OF BUCKLING

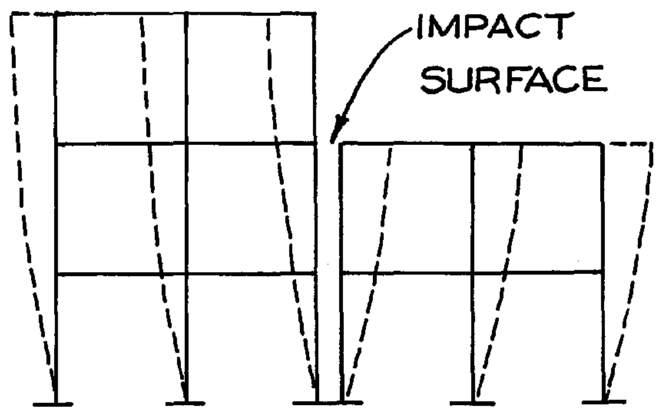


FIG. 2 IMPACT BETWEEN ADJACENT BUILDINGS

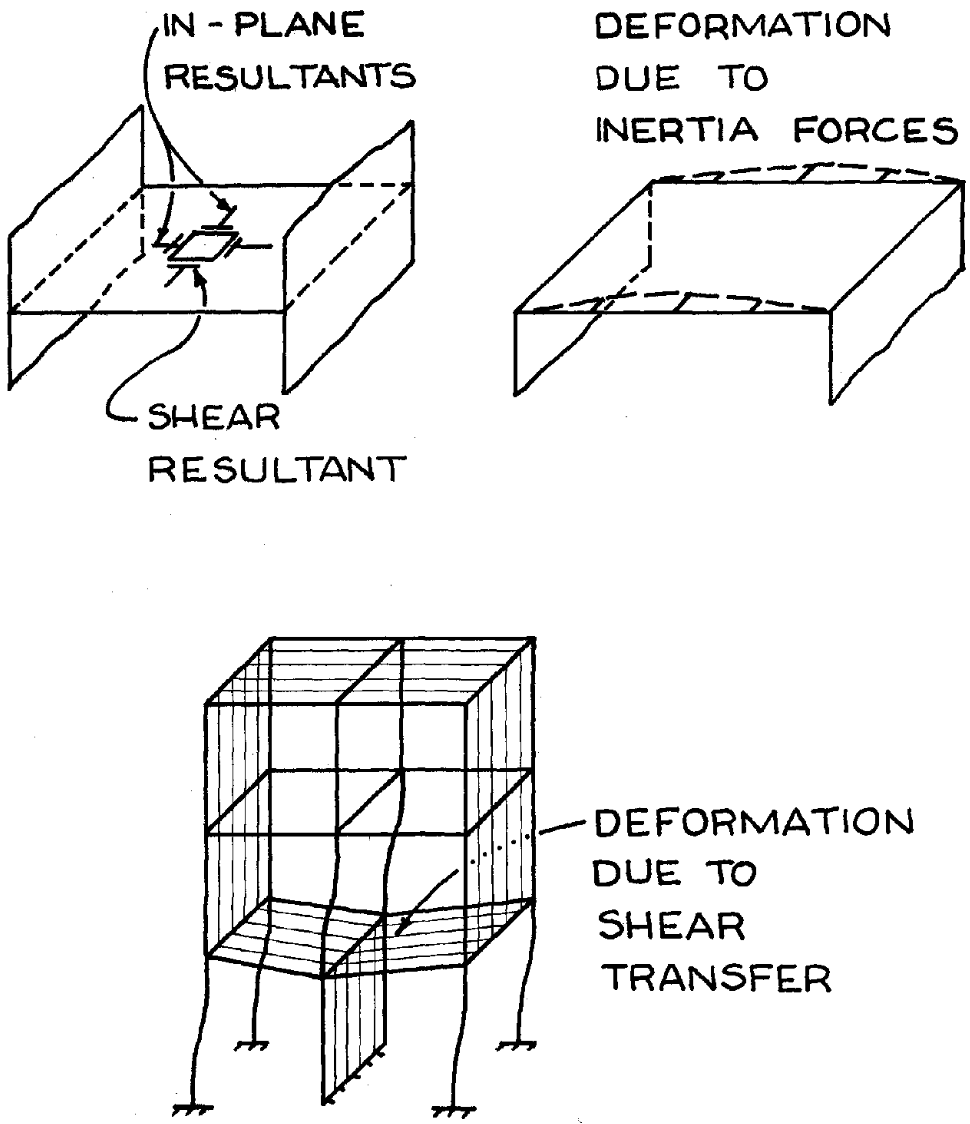


FIG. 3 - DIAPHRAGM DEFORMATIONS

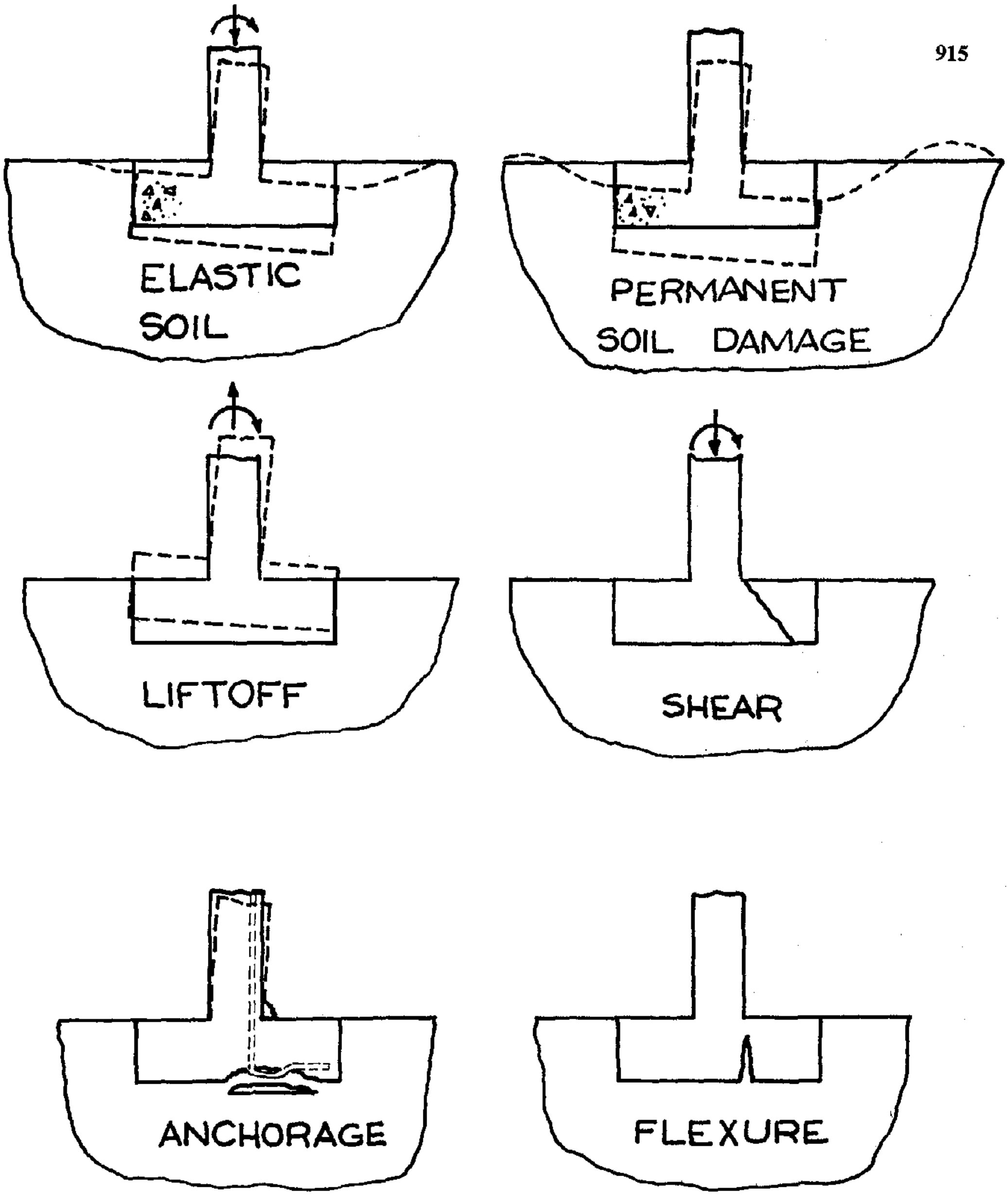


FIG. 4 FOOTING BEHAVIOR

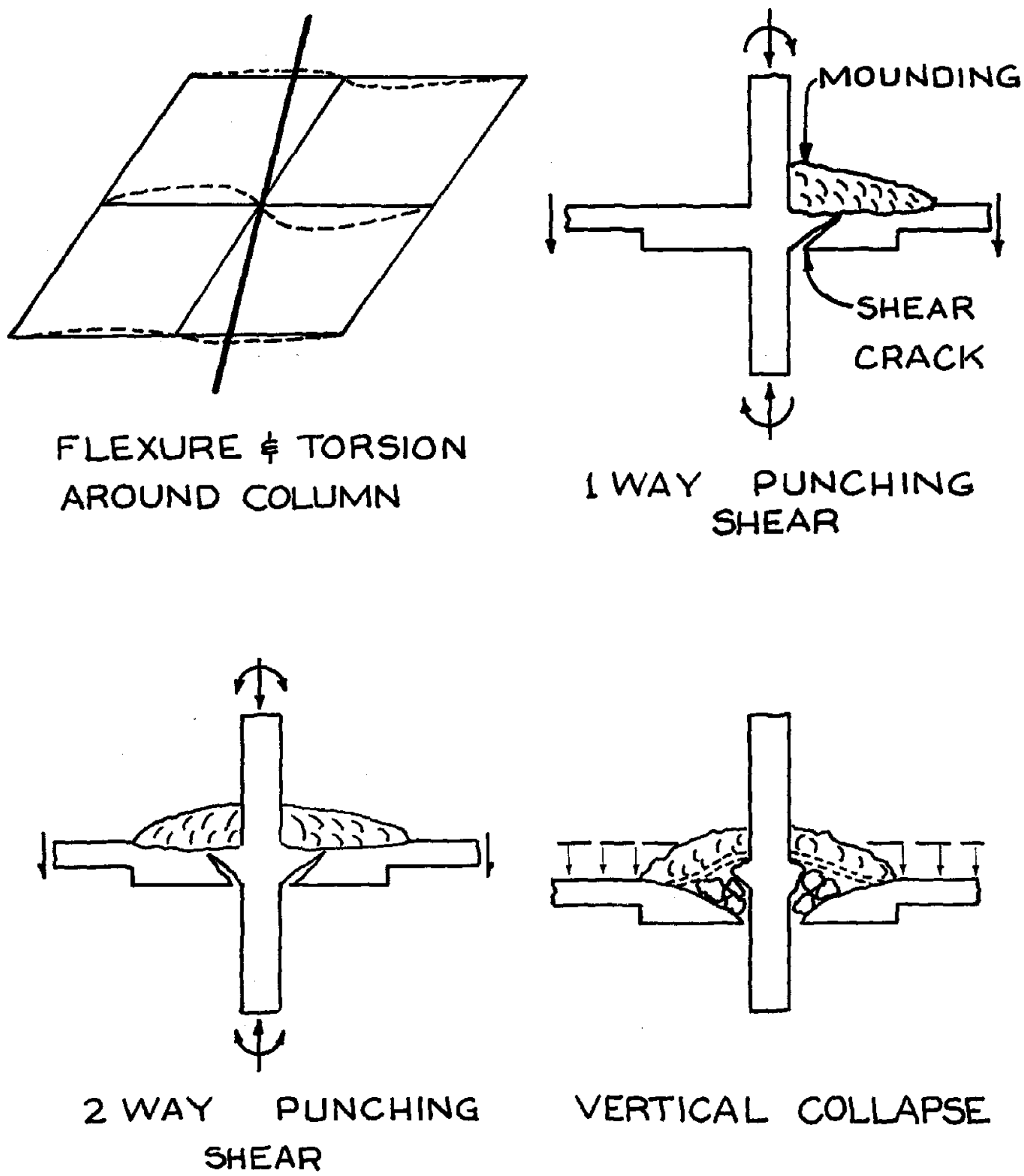


FIG. 5 - BEHAVIOR OF SLAB

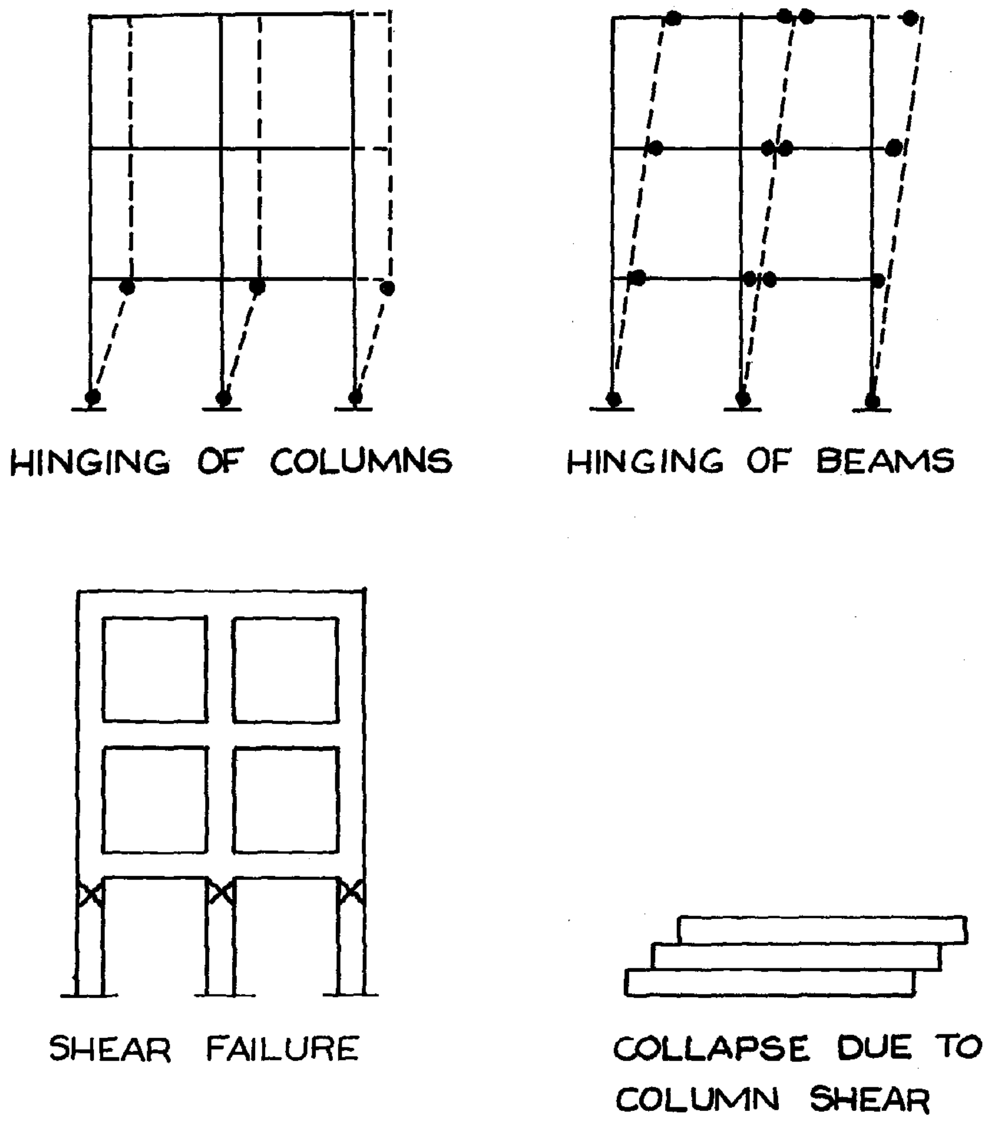


FIG. 6 - SHEAR FAILURE & HINGING

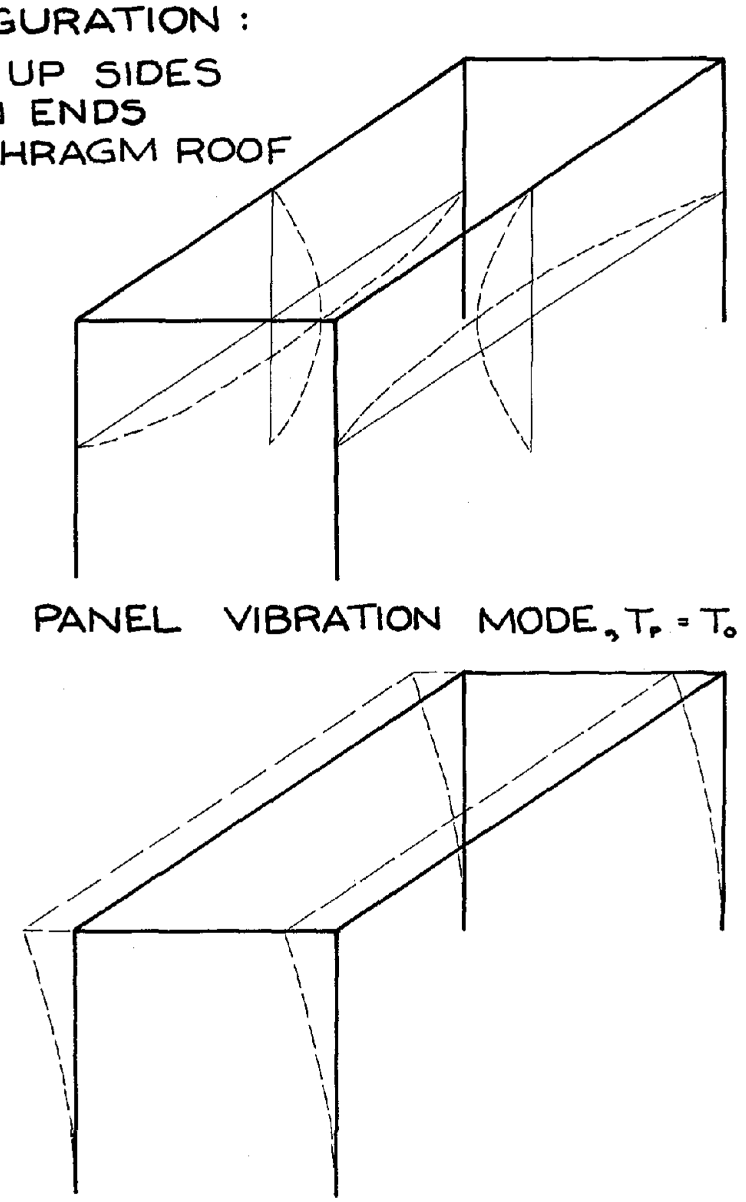


FIG. 7 - DEGENERATE MODES

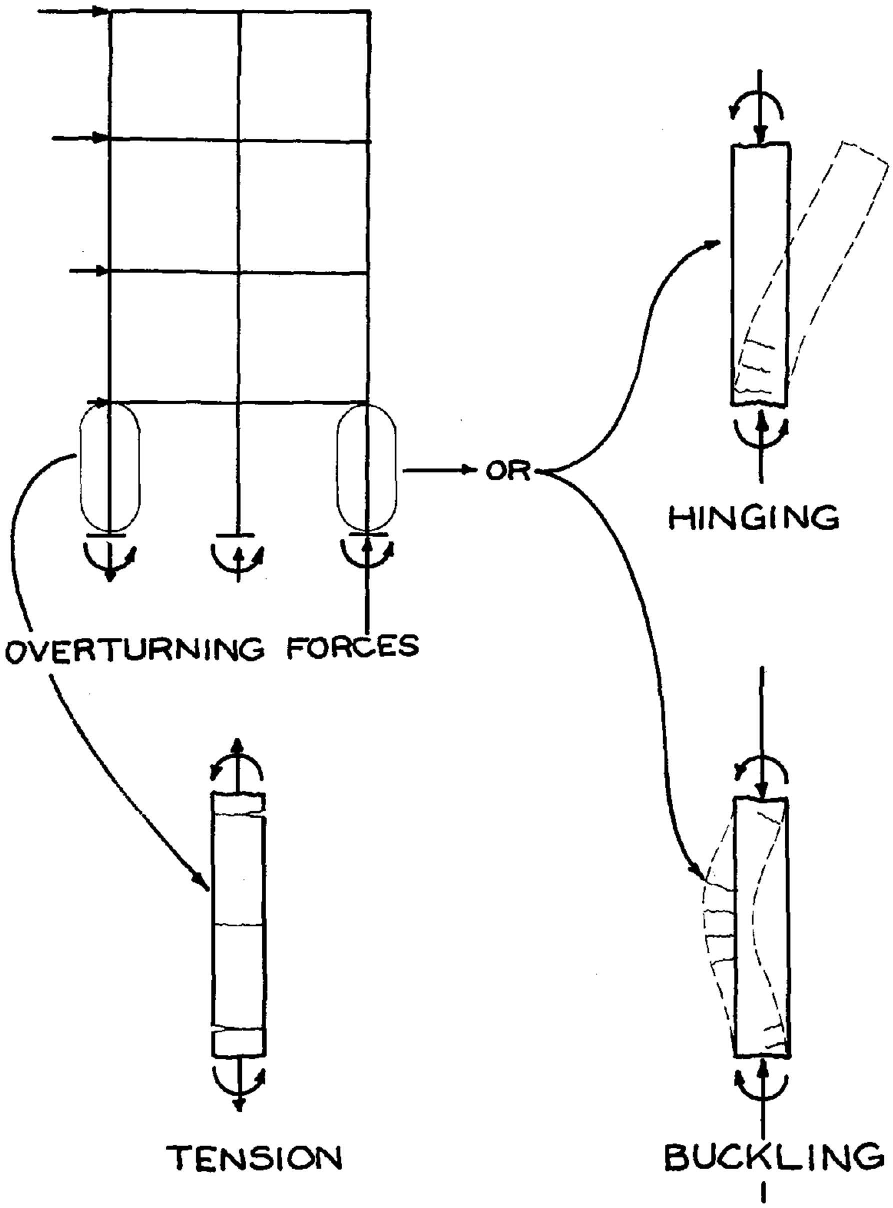


FIG. 8 OVERTURNING EFFECTS

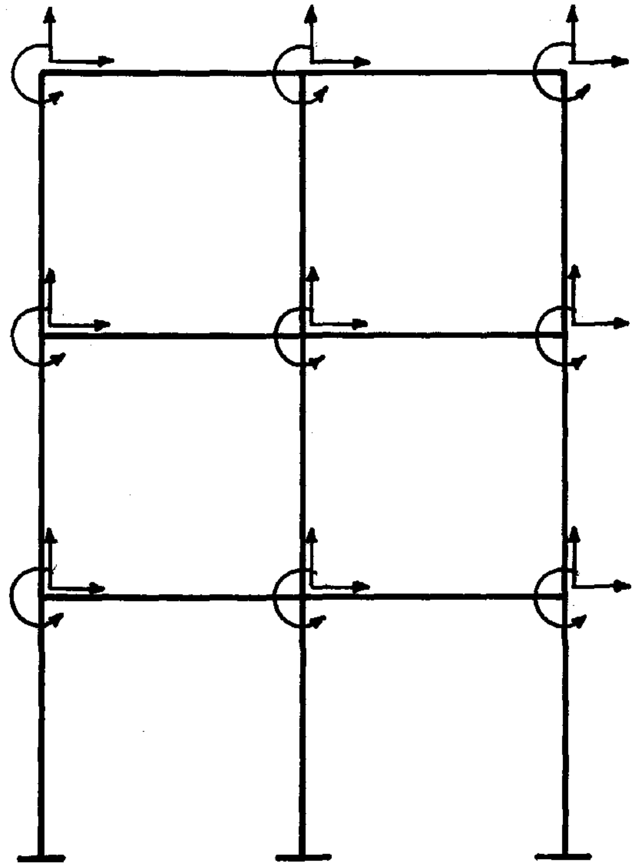


FIG.9 - 2D COORDINATE IDEALIZATION

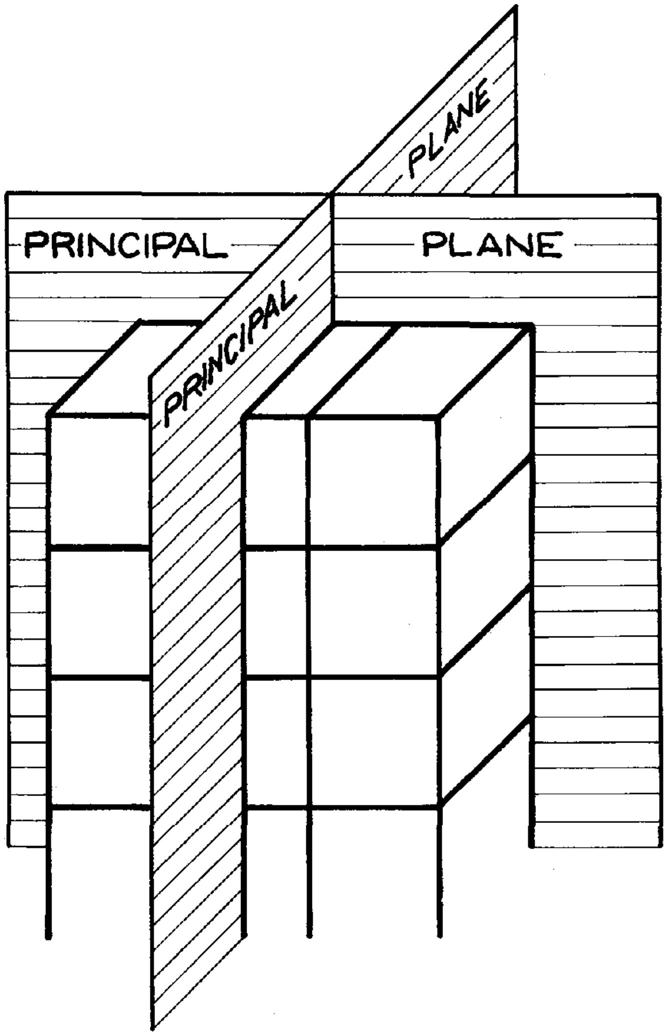


FIG. 10 - PRINCIPAL PLANES OF BUILDING MOTION

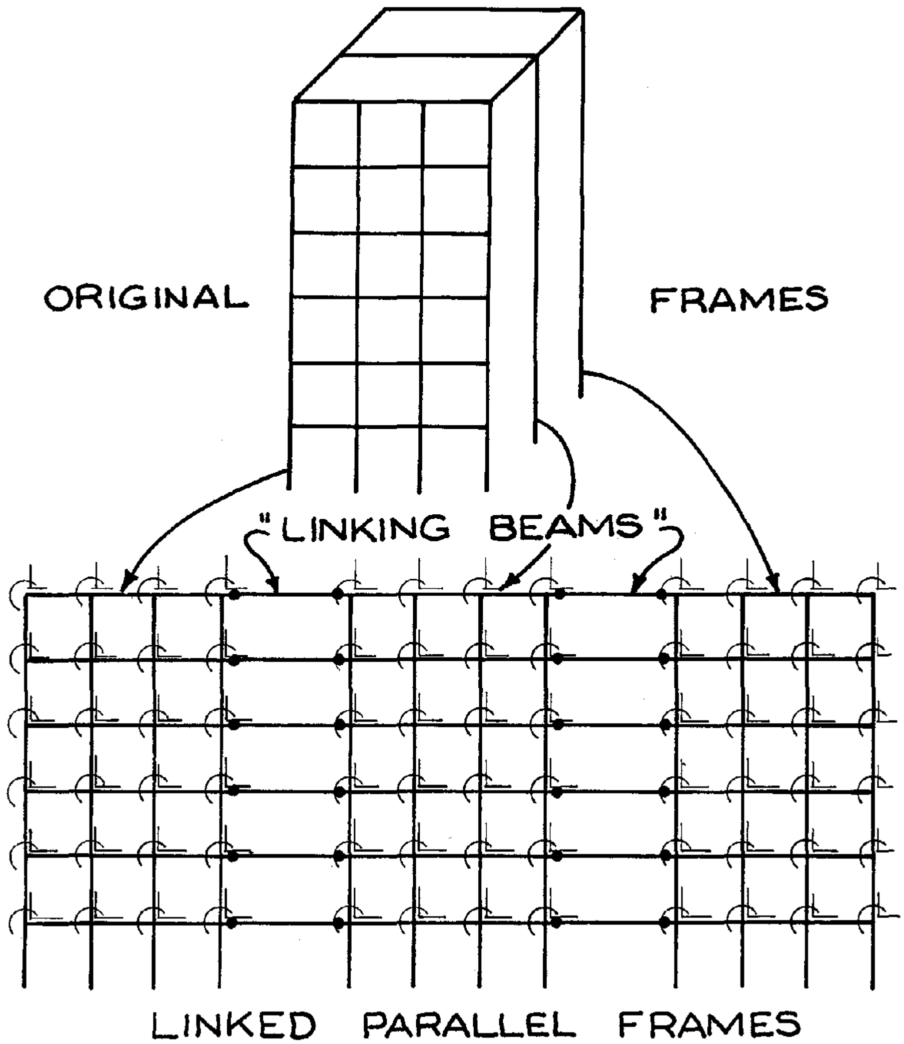
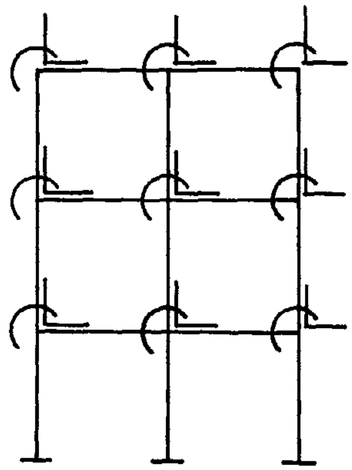
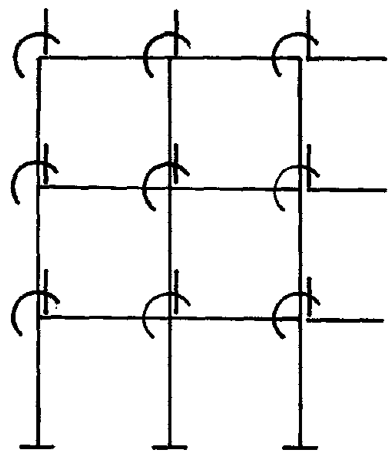


FIG. 11 - 2D REPRESENTATION OF PARALLEL FRAMES



COMPLETE 2D IDEALIZATION



RIGID FLOOR IDEALIZATION

FIG. 12 FLOOR TRANSLATION WITH ONE COORDINATE

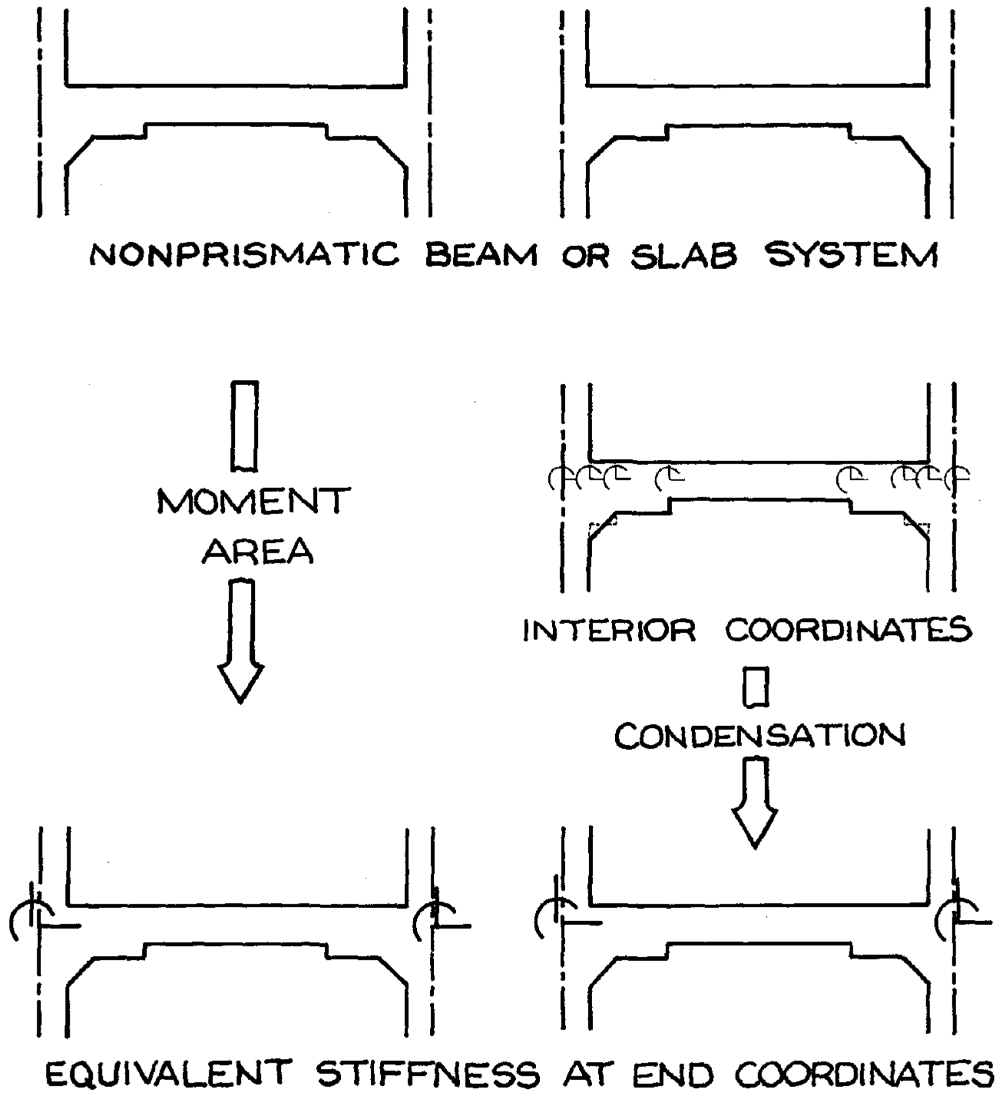
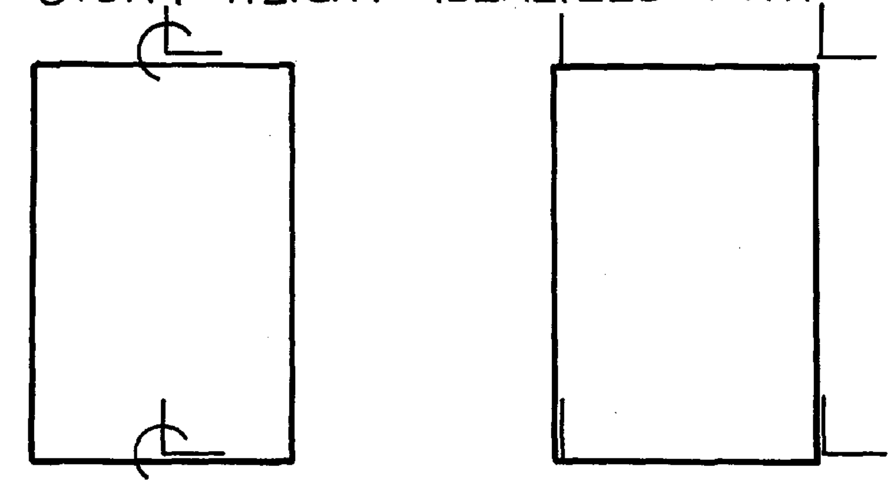
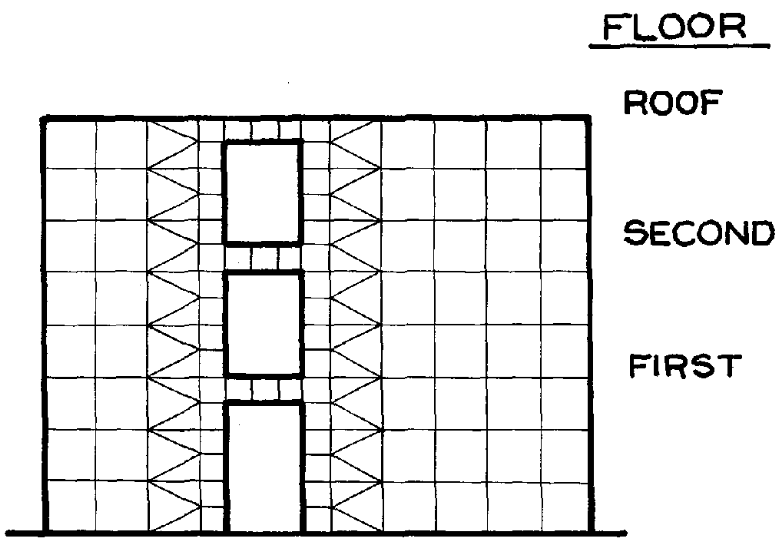


FIG. 13 - NONPRISMATIC 2D ELEMENTS

STORY HEIGHT IDEALIZED WITH

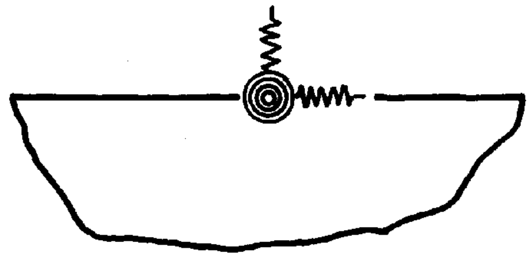


BEAM ELEMENT PANEL ELEMENT

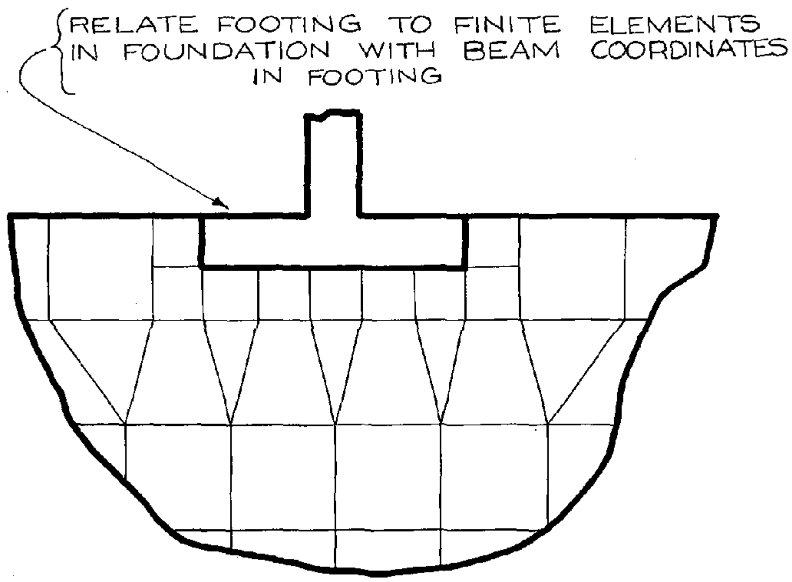


THREE STORY WALL IDEALIZED WITH FINITE ELEMENTS

FIG. 14 - FINITE ELEMENT MODELS OF WALLS



GRADE STIFFNESS MODEL



FINITE ELEMENT MODEL

FIG. 15 - FOUNDATION COORDINATE IDEALIZATION

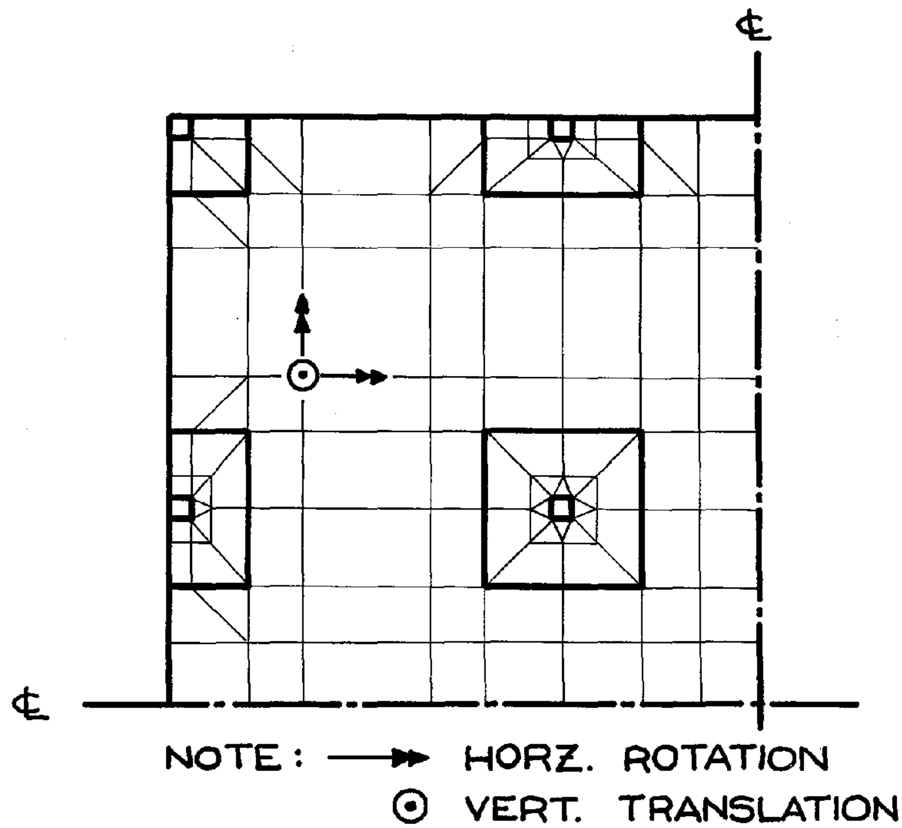


FIG. 16 - FINITE ELEMENT GRID OF FLOOR SYSTEM

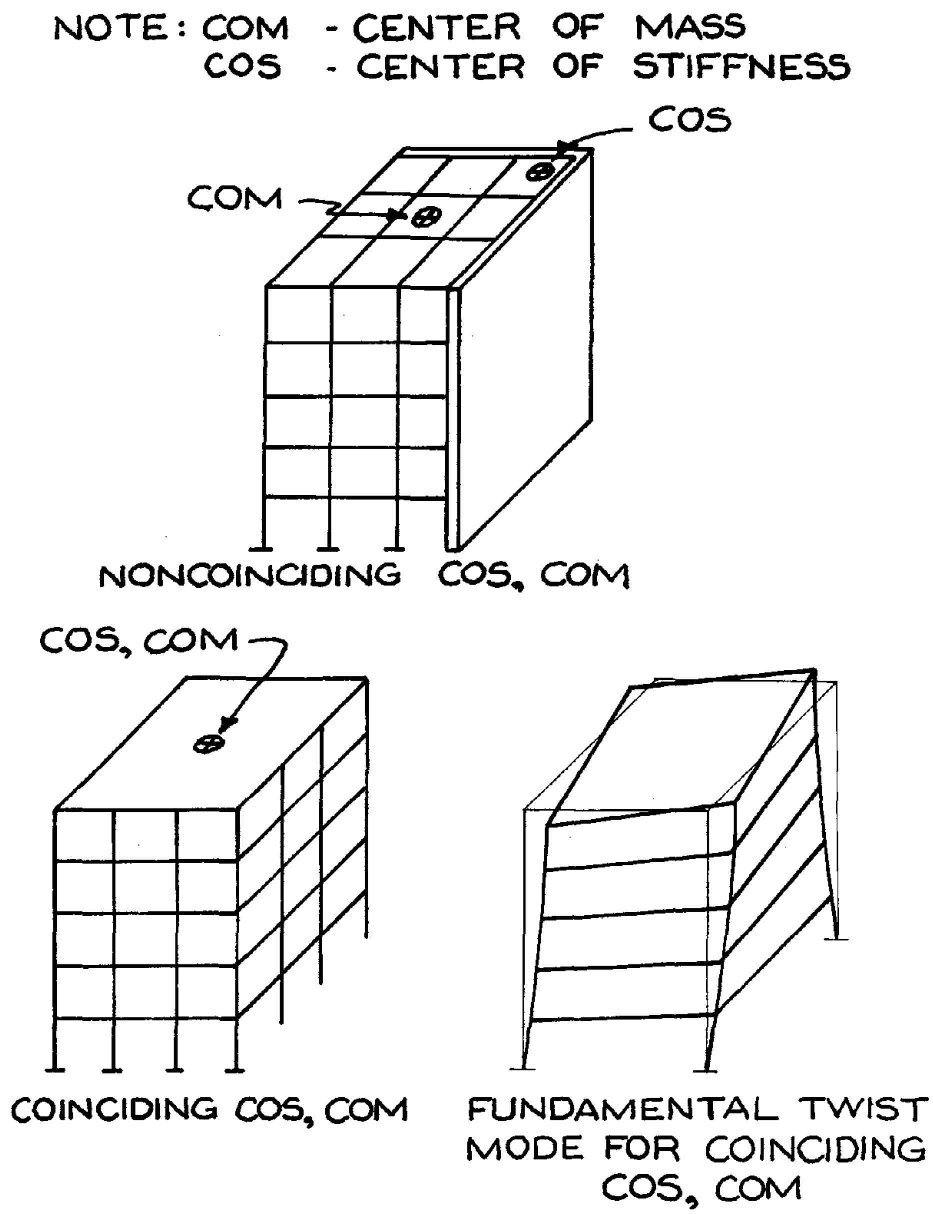
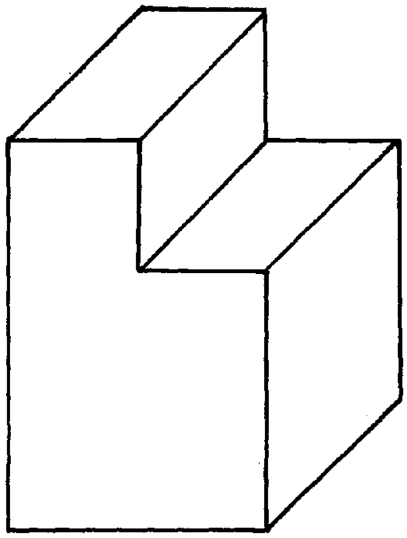
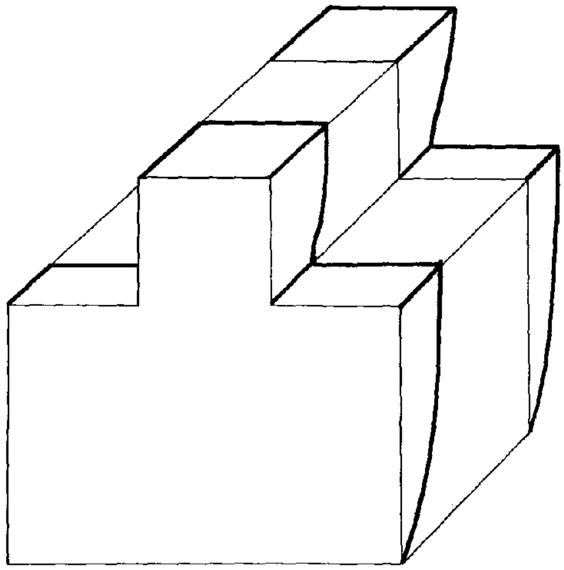


FIG. 17 BUILDINGS WITH TWIST PROPERTIES



SETBACK CAUSING TWISTING



3D RESPONSE WITH SYMMETRIC SETBACK

FIG. 18 - 3D RESPONSE DUE TO SETBACK

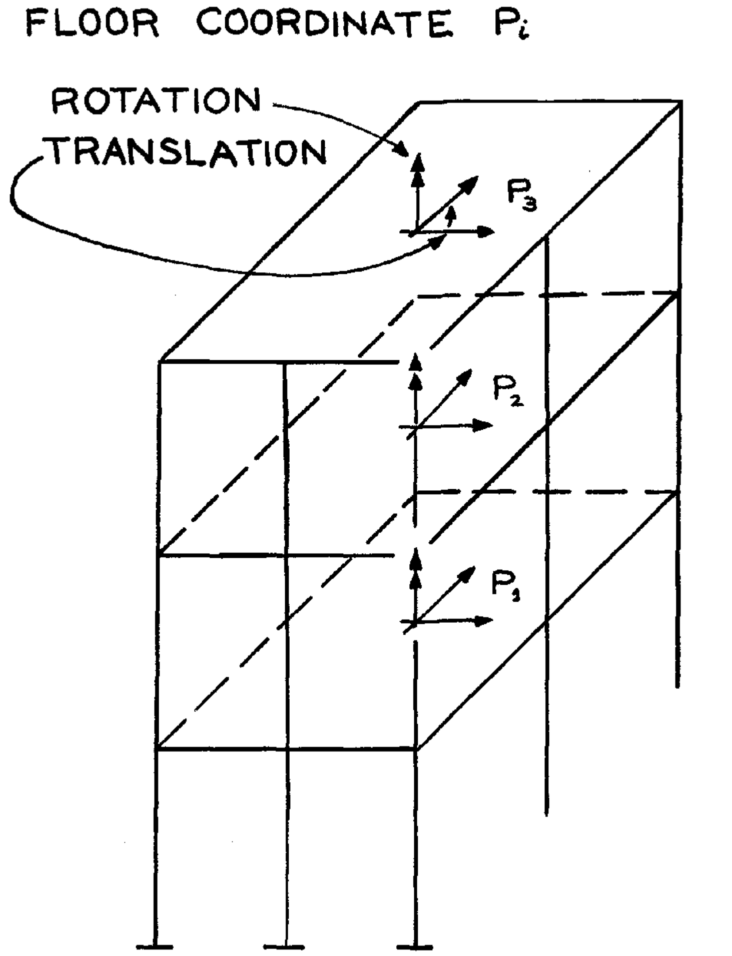


FIG. 19 RIGID DIAPHRAGM 3D IDEALIZATION

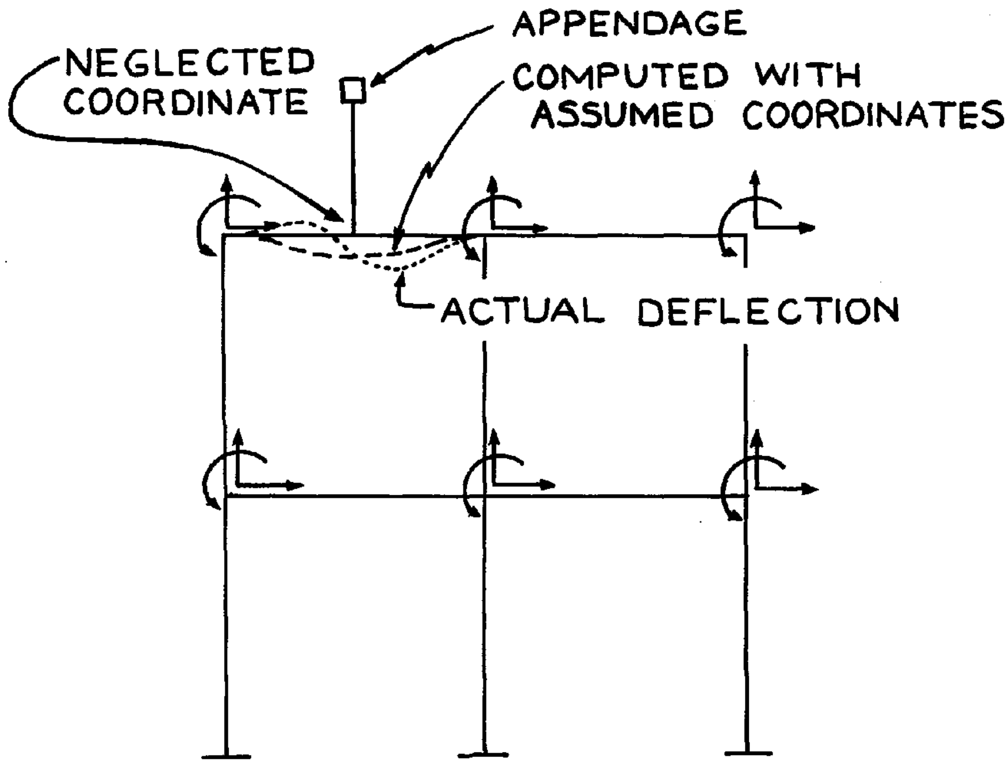


FIG. 20 OVERLOOKED BEHAVIOR PATTERN

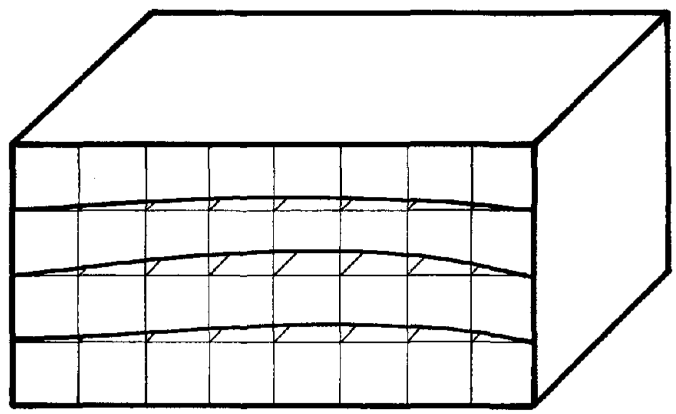


FIG. 21 - RESULTS WITH TOO MANY NODES

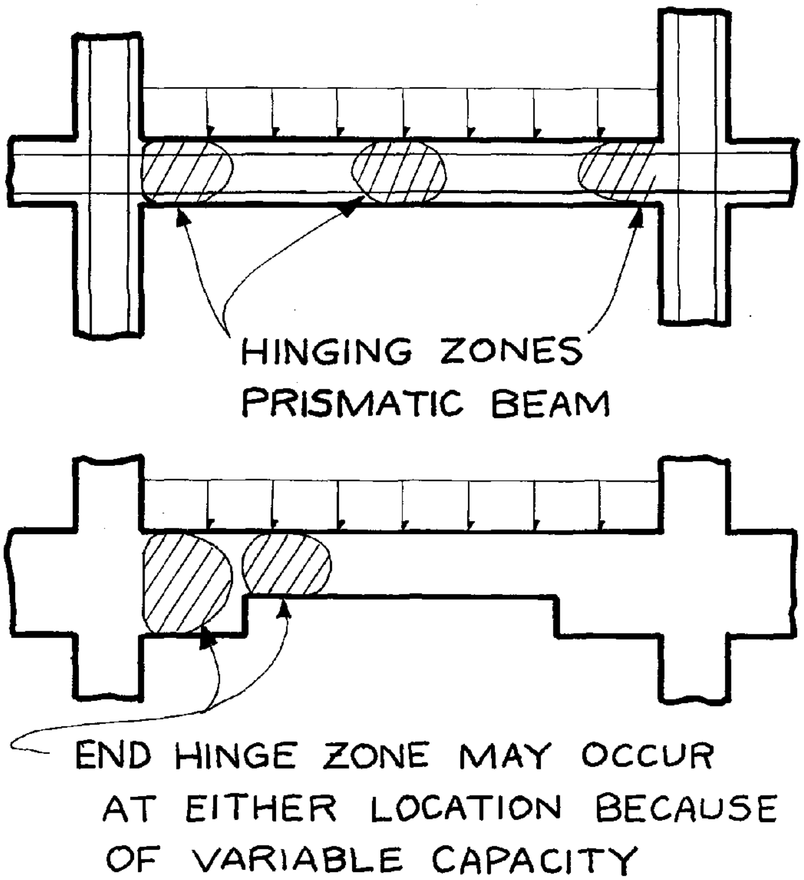


FIG. 22 - HINGING ZONES IN BEAMS

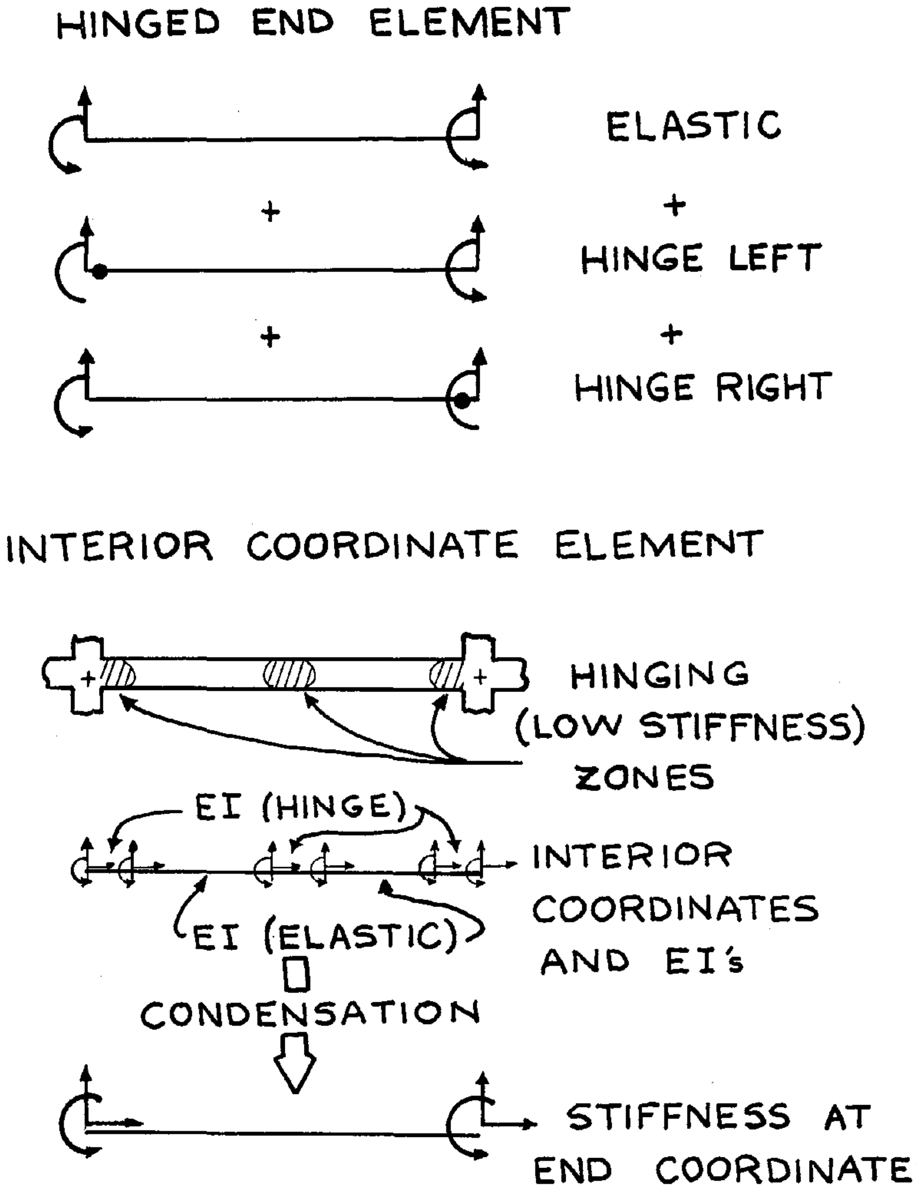
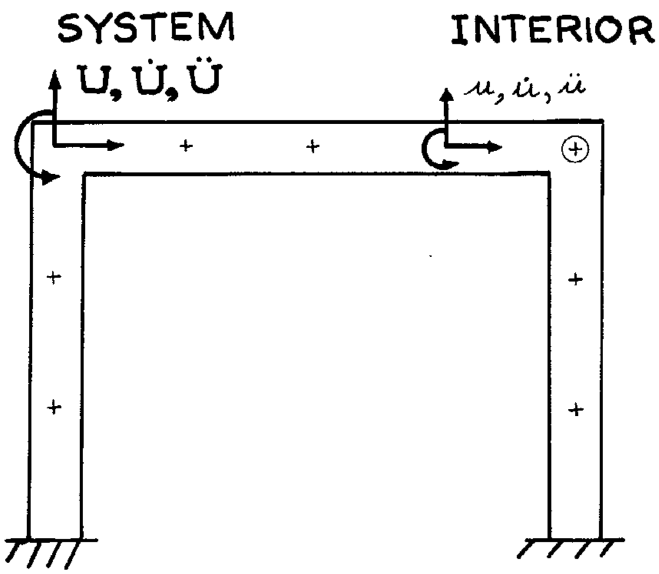


FIG. 23 - COORDINATES FOR HINGING BEAMS



FORCE BALANCE AT COORDINATES

SYSTEM - $M\ddot{U} + C\dot{U} + KU = F_{EQ}$

INTERIOR - $m\ddot{u} + c\dot{u} + ku = f_{EQ}$

FIG. 24 - EQUILIBRIUM BALANCE AT VARIOUS COORDINATES

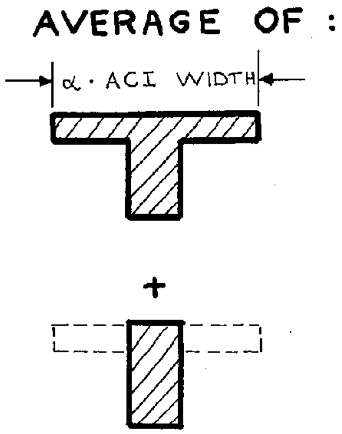


FIG. 25A - T-BEAM, TRANSVERSE BENDING

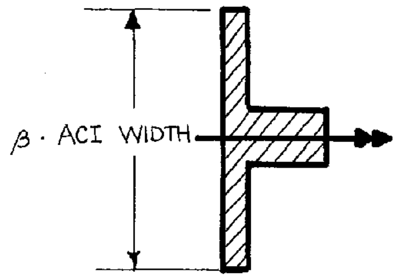


FIG. 25B - BENDING IN PLANE OF WALL, FLOOR

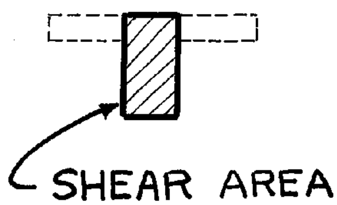


FIG. 25C - TRANSVERSE SHEAR AREA

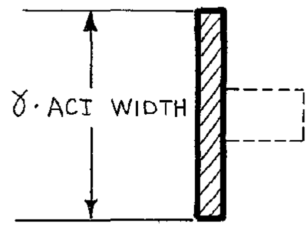
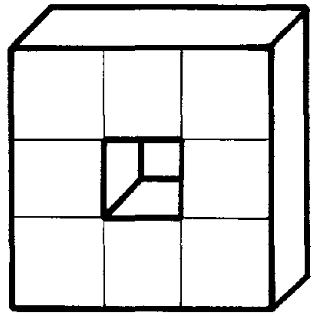
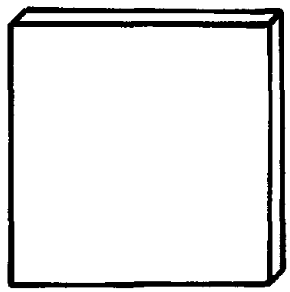


FIG. 25D - SHEAR AREA FOR DEFORMATION IN PLANE OF WALL, FLOOR

FIG. 25 - T-BEAM IDEALIZATIONS



FINITE ELEMENT GRID
OF WALL WITH OPENING



EQUIVALENT PSEUDO ELEMENT

FIG. 26 - PSEUDO ELEMENT
REPRESENTING PORTION
OF WALL

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

PROBLEMS IN THE PRACTICAL APPLICATION OF COMPUTER ANALYSIS TO REINFORCED CONCRETE BUILDING DESIGN

by

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INTRODUCTION

The development of high speed computer processing and its applications to reinforced concrete structural analysis are well known and have proven to be a valuable tool to the practicing engineer. At the same time, however, computer analysis has created a whole new set of problems that are just now being painfully learned.

These comments come from a design office environment; an environment plagued with short time schedules, limited analysis budgets, projects that change rapidly and drastically during the design phase, blessed with chief engineers who assume that all computer analysis is erroneous until proven otherwise, and founded on the motto that we build buildings, not calculations.

Time and economic restraints on design projects are a fact of life. Certainly, they were the factors that led engineers to the first applications of computer analysis programs for the solutions of basic problems traditionally done by more laborious techniques. While computer analysis provides a quick and inexpensive solution to small and simple problems, any major, complex application is forced, because of these restraints, to use course building models that leave us with beam/column models that defy their elastic assumptions and course-meshed finite element models that do only fairly well at getting around a building's square corners.

The projects referenced herein are middle-rise (6 to 16 story) concrete frame/concrete shear wall and steel frame/concrete shear wall buildings. Each project has occurred in the past five years and each has used varying degrees of computer analysis. At one extreme is an eleven-story concrete frame/concrete shear wall parking garage analyzed and designed quicky and efficiently by hand. At the other, is a fifteen-story steel frame/concrete shear wall hospital that over the course of its analysis used, misused, benefited from and found the limitations of a variety of static and dynamic computer programs.

For purposes of discussion, consider the structural analysis task for earthquake loading in terms of three phases: Overall building response, internal force distribution to the resisting elements, and local force distribution within each resisting element. In working with computer analysis in these three areas, for this class of complex structure, we have encountered serious difficulties in modeling and achieving sound results for elastic earthquake response. Elastic structural response to earthquake forces forms the foundation for all current basic and advanced methods of earthquake analysis. Even the rapidly developing methods of non-linear analysis use the basic techniques of elastic structural response within their iterative processes. For this reason, we find it important to review, strive for and insist on solutions to these modeling difficulties before their erroneous effects are lost in the magic box of high speed processing.

For convenience, consider these problems in two groups; those with understandable solutions and those solved only by intuition.

MODELING PROBLEMS WITH SOLUTIONS

Dynamic Analysis

Touted as the ultimate analysis procedure for new buildings and the most reliable measure of an existing building's lateral force capacity, dynamic analysis has been handed to the profession and the public as the answer to the earthquake problem. Unfortunately, this analysis procedure, when carefully applied can only provide a good estimation of overall building response and possibly a fair representation of internal force distribution. In our zeal to envelope the force envelopes and derive numbers far beyond the accuracy of the input assumptions, dynamic analysis denies access to a consistent, statically balanced set of forces from which a designer can study and understand a building's lateral force resisting system. As a result, we are seeing and often believing and designing for shears and moments due to fictitious hard spots, rigid diaphragms, and neglected foundation conditions, all of which can lead to conservative designs for some elements and dangerous designs for others.

The solution, obviously, is to limit the use of the dynamic analysis process to the derivation of overall building response. Furthermore, our experience has shown that given the available methods of equivalent static analysis that have all ready successfully identified and applied the critical structural parameters to an approximation of building response, we find the exercise of dynamic analysis to be, in most cases, an unnecessary process - a redundant exercise.

Torsional Effects

Engineers have long realized and understood the need for considering the effects of torsion, both in the resisting and driving sense. As a result, building code provisions require the consideration of torsion within the resisting system, define minimum torsional moments, and require the omission of any reducing effects. Obviously, no 3D building model nor currently available dynamic analysis can so thoroughly model material properties, construction tolerances, mass distributions, element stiffnesses and out-of-phase input motions to justify neglecting these code provisions. Hence, we are faced with the menacing task of distilling the actual torsional moments and resulting shears from any 3D computer analysis by considering the diaphragm rotations and building geometry, and then make the proper adjustment at the point of design. Note that attempts to adjust the computer analysis to provide designable numbers for each lateral resisting element requires multiple runs with a variety of support and load conditions.

Finite Elements

Two dimensional triangular and quadrilateral elements provide probably the best deflection and, therefore, stiffness representation for concrete shear wall and diaphragm elements. Unfortunately, their accuracy and usefulness depends highly on their number and, therefore, any analysis (especially a three-dimensional analysis with poor banding) turns into an expensive nightmare. Furthermore, because concrete design is based on forces rather than stresses, all finite element stress output must be post-processed into forces at critical sections - a simple, but very time consuming task.

Program Errors

Because of the complexity of analysis programs and the infeasibility of completely debugging any program, engineers must constantly be on the watch for invalid results due to program errors. Care must be taken to check each analysis to the point of understanding and accepting the results. Admittedly, the occurrence of a programming error is rare. However, after just four years of using a nationally promoted program, we uncovered an error in a two-dimensional frame analysis that produced a crazy set of column moments while not disrupting the external or internal static force balance. The error was traced to a newly instituted (within six months) rebanding technique.

MODELING PROBLEMS SOLVED BY INTUITION

The following four modeling areas, unique to shear wall buildings, represents a set of problems that raise serious questions about the validity of any analysis based on elastic assumptions. There are no one shot direct solutions to these problems. They are only solvable by identifying the bounds of their uncertainty and designing for the conditions in between. Such solutions require sound thinking by the designer and make any "one-model-fits-all" analysis a virtual impossibility.

Member End Joint Size

Given a concrete shear wall with a set of fairly regular openings, it is common practice to do the lateral analysis with a beam and column model. The line members normally follow the pier and spandrel centerlines and their widths are accounted for by rigid arms under the elastic assumption that plane sections remain plane. This model typically provides good internal force distribution, outputs results suitable for design, but will probably error substantially in the overall deflection and, therefore, stiffness of the wall. Such an error can have substantial effects on the internal force distribution within a 3D model.

Consider as an extreme example, the 12.2m (40') one-story wall in Figure 1. As is well known, conventional modeling techniques will result in a stiffer representation for the wall with a door than without. Obviously, the problem lies in the neglect of the shear distortion in the panel zone and thru the spandrel.

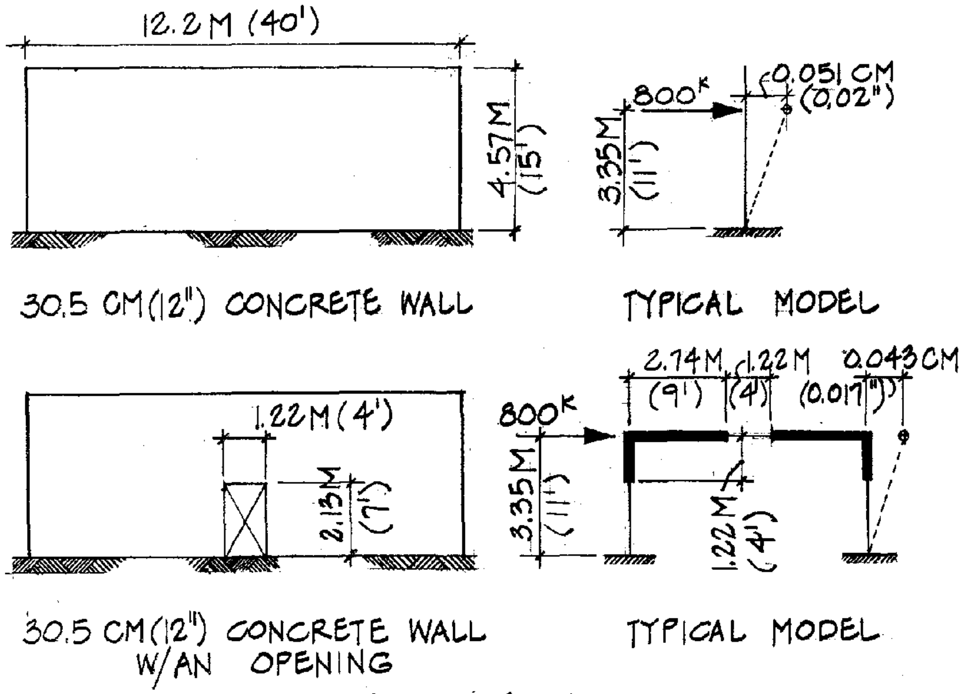


FIGURE 1

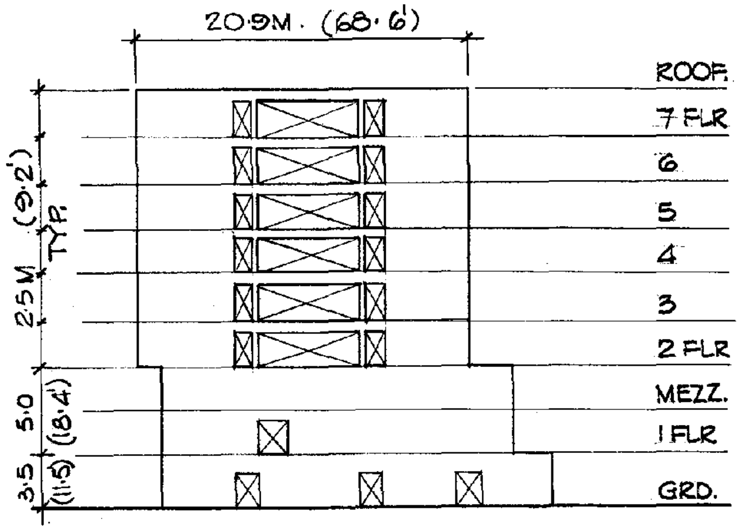
EFFECTS OF MEMBER END JOINT SIZE

Occasionally, this problem is addressed and the attempted solution involves shortening the rigid arm as much at 25%, depending on the geometry. While this adds additional softness to the system, it does so by sloppily replacing the missing shear distortion with an additional joint rotation.

Effects of Changing Modeling Techniques

More often than not, shear walls tend to have at least two patterns of openings requiring a change in modeling and a point of transition. This follows since few buildings support the same function on all their floors.

Consider the concrete shear wall shown in Figure 2. This wall resisted, along with fourteen other shear walls and concrete frames, the transverse lateral loads from a 24.4m x 61m (80'x200') eight-story concrete hotel. The other lateral elements varied from near solid walls at the ends of the structure, to flexible concrete frames. Four of the fourteen walls had large changes in stiffness at the second floor similar to the wall shown.



CONCRETE SHEARWALL

FIGURE 2

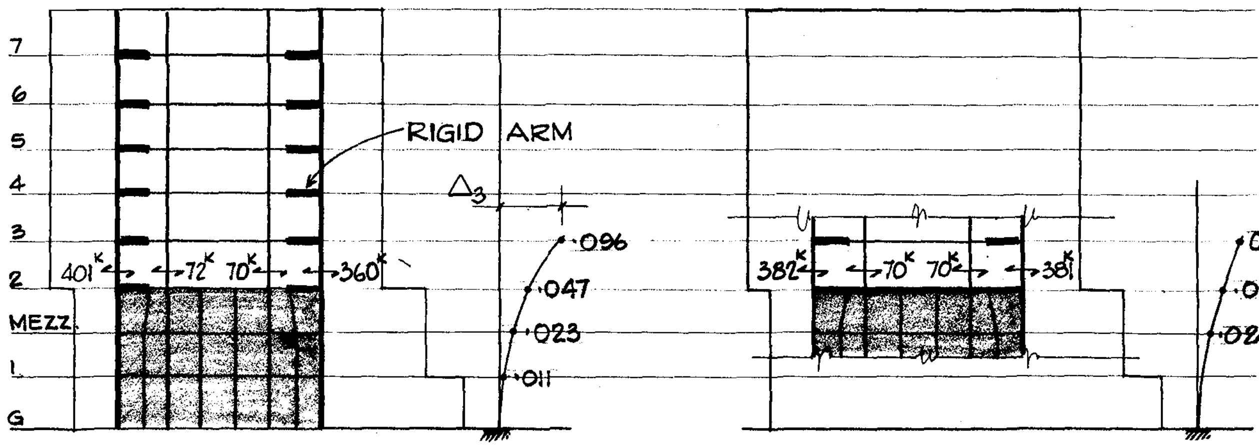
EFFECTS OF CHANGING MODEL TECHNIQUES

The basic model of the wall included a beam and column model in the upper level and a column and finite element model in the lower levels. Four models, all based on common assumptions were considered at the transition zone. The results, as shown in Figure 3, produced a 40% variance in stiffness (measured at the third floor) and a 100% variance in shear distribution to the second floor piers. Note that none of the models produced consistently conservative results for all local elements.

Diaphragm Flexibility

Engineers have, on numerous occasions, witnessed diaphragm damage from earthquake loading in the form of shear, chord and collector failures. Traditionally, concrete diaphragms in concrete shear wall buildings have been assumed to be rigid allowing for the internal force distribution to be based only on the relative rigidities of the lateral force resisting elements.

As is well known, a rigid diaphragm assumption works well in buildings with regularly placed, full height shear walls of fairly uniform stiffness, and must be supplemented with flexible diaphragms in areas of major discontinuities or overall shear transfers.



MODEL-TYPE 1

ASSUMPTIONS

1.) EXTERIOR PIERS REMAIN PLANE AT TRANSITION

2.) COLUMN BETWEEN OPENINGS ACHIEVES BASE FIXITY THRU CONNECTIONS AT LEVEL 2 AND MEZZ.

MODEL-TYPE 2

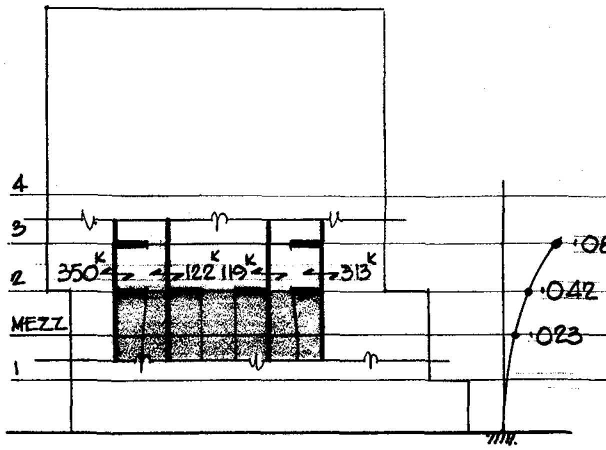
ASSUMPTIONS

1.) SOLID PART OF WALL REMAINS PLANE

2.) COLUMN BETWEEN OPENINGS FIXED AT TOP OF WALL

FIGURE 3

EFFECTS OF CHANGING MODEL TECHNIQUES

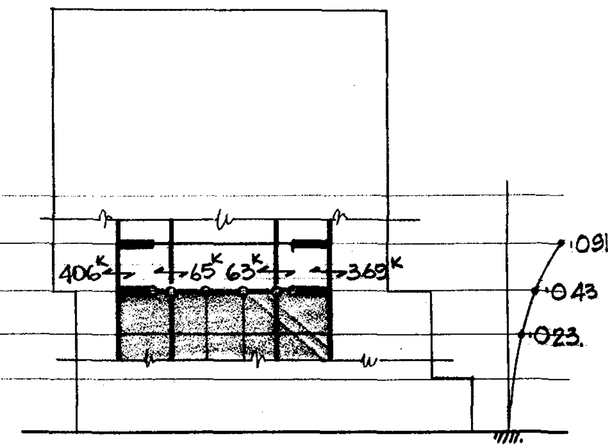


MODEL-TYPE 3

ASSUMPTIONS

1.) EXTERIOR PIERS REMAIN PLANE AT TRANSITION

2) COLUMN AT OPENING ACHIEVES FULL FIXITY AT TOP OF SOLID WALL



MODEL-TYPE 4

ASSUMPTIONS

1.) EXTERIOR PIERS REMAINS PLANE AT TRANSITION.

2.) RIGID TRUSS LINKS PROVIDE EQUAL DEFFLECTION. ALONG TOP OF WALL - PREVENT DISTORTION.

3.) COLUMN BETWEEN OPENINGS ACHIEVES FIXITY AS IN MODEL TYPE - 1

FIGURE 3
(CONTINUED)
EFFECTS OF CHANGING MODEL TECHNIQUES

Unfortunately, when comparing the results of a recent rigid diaphragm analysis with a subsequent flexible diaphragm analysis, when only the properties of the diaphragms were changed, we found substantial changes in shear distribution from very small diaphragm distortions.

Consider the model used for the upper stories of a fifteen-story steel frame/concrete shear wall hospital. In the upper floors (8-15), the building consists of a 23.5m x 64.3m (77'x211') floor of metal deck with concrete fill with four main transverse and three longitudinal shear walls. At the seventh floor, the building grows into an "L" shape that includes the extension of one transverse shear wall and the addition of a fourth longitudinal wall. See Figure 4.

A three-dimensional model, with a mathematically rigid diaphragm, and equivalent columns for shear walls produced a 2000^k shear transfer under longitudinal lateral forces to the new shear wall at the seventh floor, while creating a shear reversal in the two short longitudinal shear walls. A three-dimensional model composed of the same equivalent columns linked by a finite element flexible diaphragm model that included all the structural steel chords and the stiffening effects of the shear walls produced only a 240^k shear transfer to the new wall and no shear reversal in the others. This result is no surprise and is, in fact, what an experienced engineer would have expected and designed for without a computer analysis. The surprise came from the amount of diaphragm distortion in the flexible model.

The maximum flexible diaphragm distortion from the rigid diaphragm position was a little more than 5/16" - far less than the deflection it would take to close the allowable shrinkage cracks. This certainly raises serious question about the rigidity of any concrete diaphragm working in a shear wall system, and warrants a serious review of one of the most common analysis assumptions made today.

Foundation Conditions

Concrete shear wall buildings, by nature, localize lateral force resistance within a few elements. Computer models based on full foundation fixity require near impossible foundation rigidities to provide the needed resistance for the overturning moments. They also drastically underestimate the period of the building, disrupt the internal force distribution within the building, and the local force distribution within the lateral elements.

Consider the coupled shear wall in Figure 5. This wall worked with similar walls in a three wall transverse resistance system for a 21.9m x 60m (72'x197') steel frame/concrete shear wall building. Because of the perfect opening regularity, the wall was analyzed with a beam and column model with appropriate adjustments made for the member end joint sizes. Because of the sensitivity of coupled shear walls to foundation conditions, three models were studied with a variety of foundation conditions. As shown in the figure, the overall stiffness of the wall varied in excess of 100% as did the spandrel shears. Note also that none of the models gave consistently conservative results.

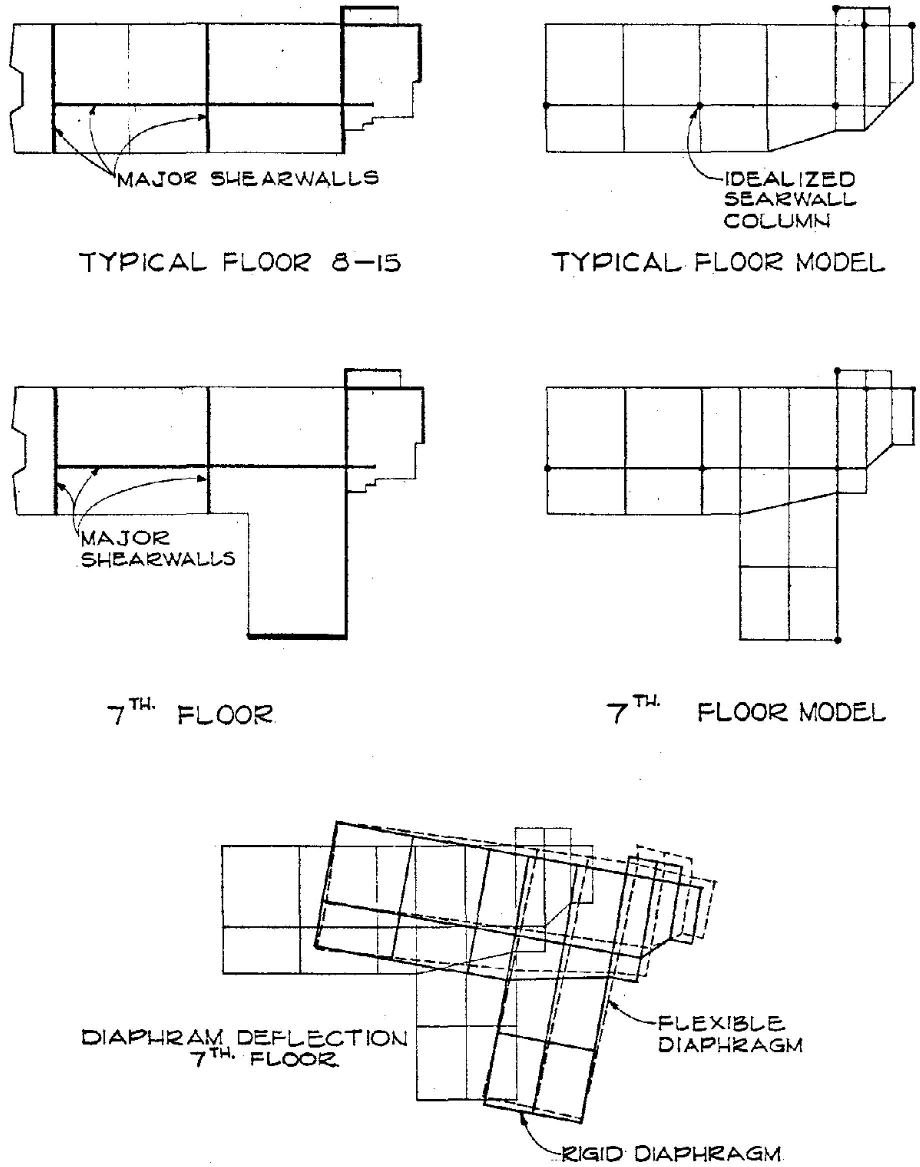
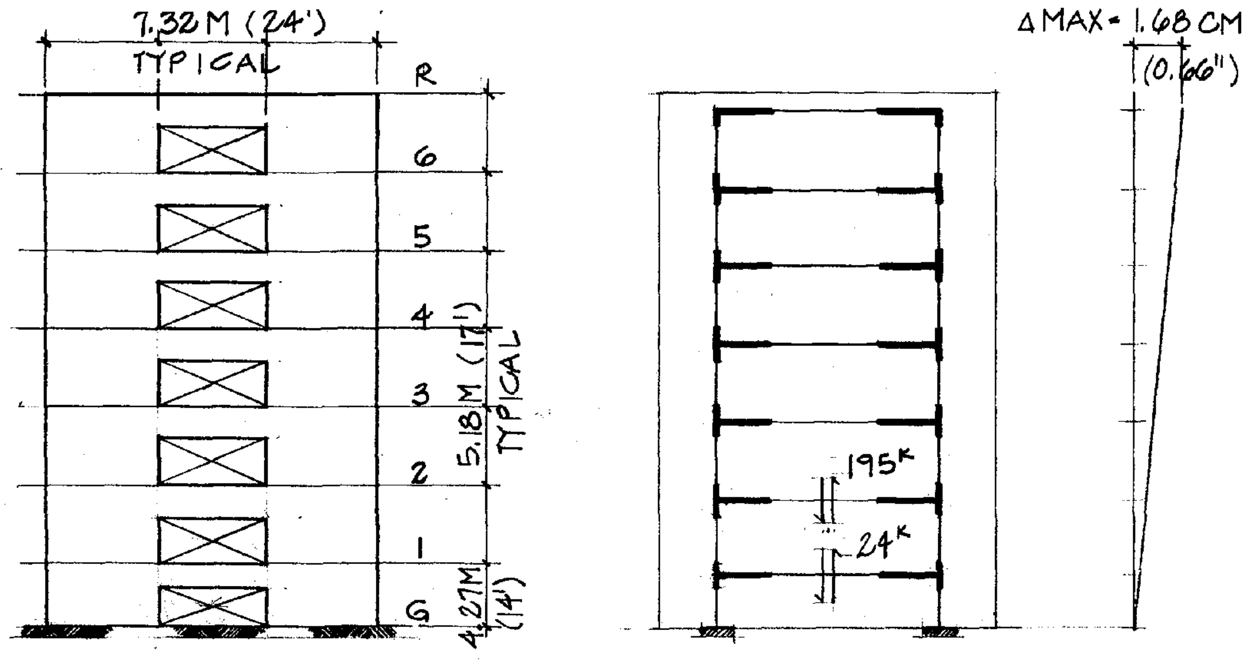


FIGURE 4
EFFECT OF DIAPHRAM FLEXIBILITY



CONCRETE SHEARWALL   MODEL TYPE #1

FIGURE #5
EFFECT OF FOUNDATION CONDITIONS

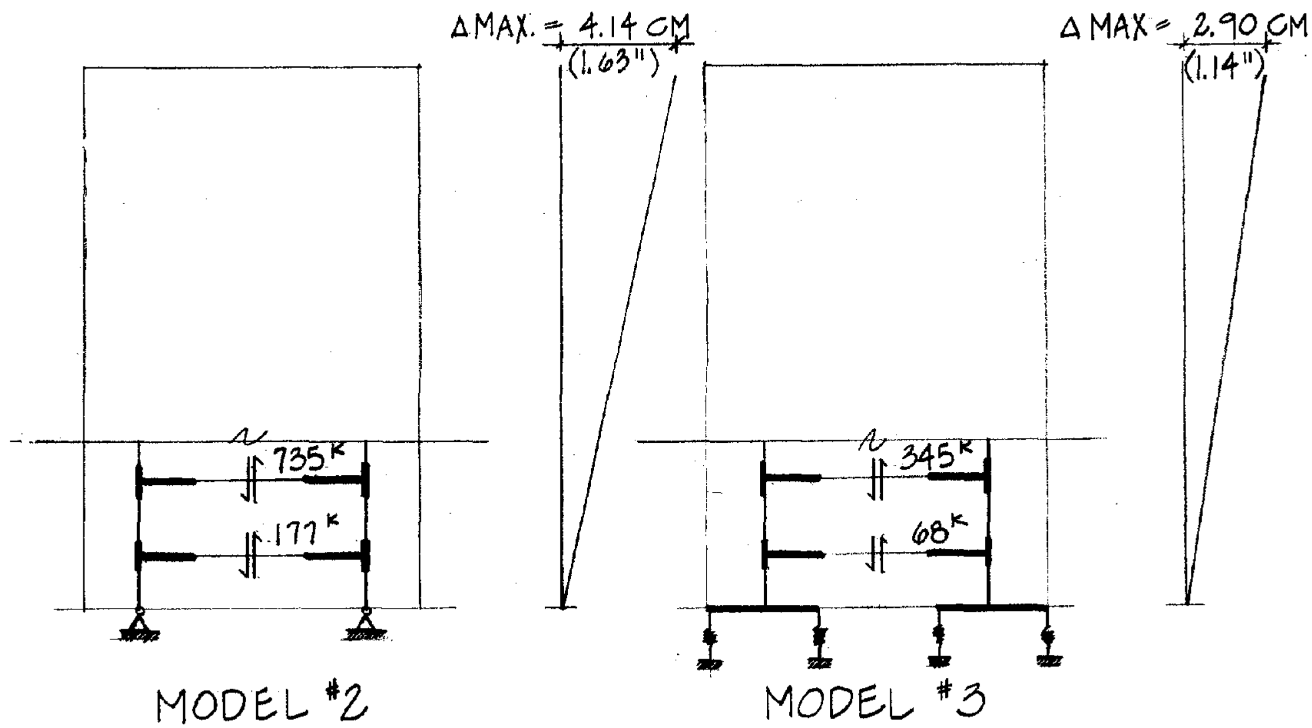


FIGURE #5
(CONTINUED)
EFFECTS OF FOUNDATION CONDITIONS

The overall effects of the above mentioned modeling problems obviously depend on the use of the models and the building geometry. As long as each of the problems is isolated, studied and resolved, a valid analysis is possible. However, if they are allowed to go unresolved, and allowed to interact within a dynamic analysis, or at the internal force distribution level of static analysis, the results will be the development of fictitious hard spots resulting in shear transfers within the building requiring wastefully conservative design in resisting and delivering those shears while at the same time providing for nonconsecutive design in the unloaded areas.

CONCLUSIONS

It has been the intent of this paper to review the serious modeling problems inherent in computer analysis of complex structures for elastic earthquake response. No reference has been made to specific programs and none is intended. These comments are valid for any analysis based on elastic assumptions.

Two sets of problems have been assessed. Those with solutions and those without. Under problems with solutions, dynamic analysis as the ultimate solution, the nuisance problems of adjusting for torsional effects, the extra work of finite element models and pitfalls of programming errors have been iscussed. Under problems without solutions, those solved only by parameter tudies and intuition, the dilemmas of specifying member end joint sizes, hanging modeling techniques within a lateral resisting element, diaphragm lexibility, and foundation conditions have been reviewed and demonstrated.

Obviously, considerable work needs to be done in studying and reviewing he critical parameters involved in each of these difficult areas. Such an ffort might lead to the development of modeling standards that will speed up he process of creating usable computer models. Certainly, the application f our current modeling techniques to the total analysis of complex structures oes not provide the needed accuracy to support the advanced techniques of tructural analysis, including non-linear analysis.

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

EFFECTS OF TWO DIMENSIONAL EARTHQUAKE MOTION ON RESPONSE OF R/C COLUMNS

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INTRODUCTION

Economical design demands that structures be designed to deform into the inelastic range when subjected to strong motion earthquakes. In the inelastic range, the six components of ground motion (three translational and three rotational components) acting simultaneously produce a complex interaction in the structural response. The columns in a space frame are a good example. Even if consideration is restricted to the two translational components of motion in the horizontal plane, the columns are subjected to biaxial bending. The axial force also varies to some degree due to the overturning moment. Analytical response studies often deal with only a single component of ground motion. Likewise, seismic design procedures usually require only that the effects of a single horizontal component acting in various directions be considered (in addition to effects of vertical motion). Our understanding of the effects of inelastic action and as a result code-specified levels of resistance are based to a large extent on uni-directional response studies. It is important therefore to know whether or not two- or three-dimensional excitations produce significantly larger displacement responses.

Several recent analytical studies [1, 5-9, 11-13] have been directed toward this end. The great majority of these studies have dealt with single mass systems subjected to two horizontal components of ground motion, in an effort to gain a basic understanding of the phenomenon and to identify important parameters. Results for multistory systems are very meager. The objective of this paper is to describe and rationalize the general features of two dimensional (2D) response for single mass systems using both previous and new results [9] for illustrative purposes.

COMPARISON CRITERIA

The most productive way of gauging 2D response is to relate it to one-dimensional (1D) response, about which much is known. In doing this, it is convenient to recognize two distinct effects. The first, which might be called a "correlation" effect, exists even in elastic systems. In an elastic system, two orthogonal components of input to a single mass system produce two uncoupled orthogonal components of response. The maximum (vectorial resultant) displacement is greater than or equal to the maximum response in either component direction. However, since the input motions are for practical purposes uncorrelated [3], the maximum responses in the component directions are very unlikely to occur at the same time. It is undoubtedly this rationale which forms the basis for code provisions. A limited amount of data indicates that this effect alone accounts for only about a 10 per cent increase in maximum displacement in both the elastic and inelastic ranges. The second effect might be called an "inelastic interaction" effect. It results from a change in the resistance properties of the structure due to 2D motion. For a reinforced concrete column, for example, the moment-curvature relations for biaxial bending can be significantly different than those for uniaxial bending.

The response parameter of greatest interest is the maximum displacement response normalized by the "yield" displacement. This normalized maximum displacement will be referred to as "ductility." In comparing 1D and 2D response, most previous investigations have attempted to isolate the inelastic interaction effect. Since it is the combination of the correlation and inelastic interaction effects which is of importance for design, the response comparisons in this paper are made in terms of vectorial resultants for 2D response and the largest of the two individual components for 1D response. This also appears to produce more consistent trends in the results.

EFFECTS OF 2D MOTION

General Description

In general terms, yielding of a dynamically excited single degree of freedom system with a softening but nondeteriorating resistance curve may be thought of as producing an equivalent linear system with longer period and higher viscous damping. This is a well known concept, and when it is used in conjunction with smoothed elastic response spectra it is useful in rationalizing important aspects of 1D inelastic response.

When a system is subjected to 2D excitation, it yields at an earlier stage in the response, and thus the effective period shift tends to be larger than for 1D response. The effective damping for 2D response may be about the same as for 1D response or it may be significantly less depending on the hysteresis characteristics of the material. Relating this information back to a smoothed elastic response spectrum, it would be expected that if the system elastic period is such that an increase in period produces an increased displacement response, 2D motion would be amplified compared to 1D response. This should be most pronounced for systems with low elastic period, in the range of 0.2 to 0.5 sec. The fundamental elastic periods of many low-rise structures are in this range. Results will be presented in the next section which show that a significant increase in response can occur for longer periods as well, so that a criterion in addition to elastic period is needed to define the possible problem area. Once the displacement response is increased, the effect of gravity aggravates the situation, and creates the possibility of collapse. The duration of strong ground motion then becomes an important factor [2, 10].

Results

No attempt is made to present a comprehensive set of numerical data. Rather, representative results are selected from several different studies to illustrate the general trends discussed above.

The results are all for single mass systems supported on a single fixed-fixed circular column. The variables considered are earthquake strength (relative to system strength), elastic period, gravity load, type of resistance curve, and type of earthquake input (as characterized by several different recorded accelerograms). The relative earthquake strength is measured by the ratio U_E/U_Y where U_E is the largest of the two components of elastic response of the system (for the same viscous damping as used in the inelastic response calculations), and U_Y is the system yield displacement. When a log-log plot is made with U_E/U_Y as abscissa and U_M/U_E as ordinate, where U_M is the maximum displacement response, then lines of constant ductility, U_M/U_Y, are inclined at 45 degrees. The ordinate U_M/U_E is a measure of intensity of output response relative to input. 1D elastic system response plots as a horizontal line $U_M/U_E = 1$. 2D elastic system response plots as a horizontal line $U_M/U_E > 1$, reflecting the "correlation" effect. Elastoplastic responses approach these values as U_E/U_Y approaches 1, since the system then remains elastic. For more complex resistance curves, the force displacement response may not be linear up to U_Y, so a value $U_E/U_Y \leq 1$ does not imply elastic response.

Fig. 1 shows results [8] for an elastoplastic system (elastic period 0.3 sec, 1% damping) subjected to the Pacoima (1971) accelerograms. Gravity (P-delta) effects are not included.

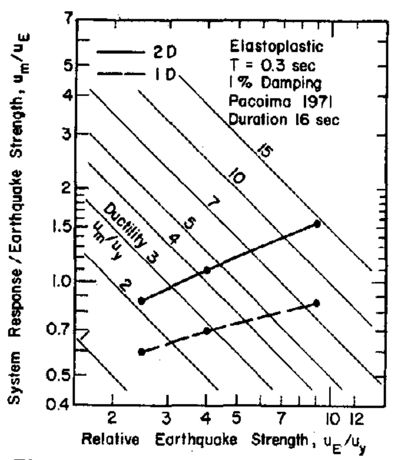

Fig. 1 Response Ductilities in Elastoplastic System

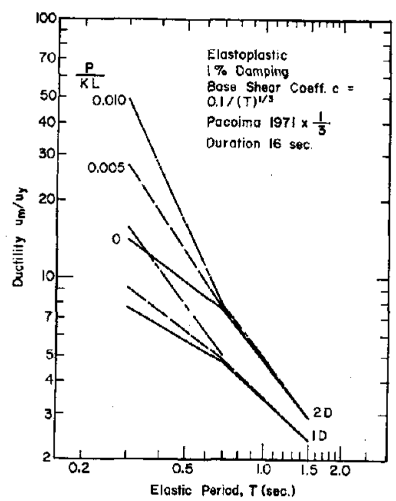

Fig. 2 Gravity Effect in Elastoplastic System

In this particular case, the ratio of 2D to 1D response increases gradually with earthquake strength and ranges from about 1.5 at a 1D ductility of 2 to about 1.8 at a 1D ductility of 7. Obviously the precise numerical values depend on many factors, such as peculiarities of earthquake input, and elastic period and hysteresis properties of the system. The uniform trend shown is probably also due to the fact that each curve is drawn through only three data points. In fact, at low values of U_E/U_Y the magnification is often not as large as shown in Fig. 1. Nevertheless, it is true that response magnifications up to about 2 often occur. Studies of elastoplastic and bilinear systems [5,6,8] have tended to indicate that these amplifications occur only for short period systems, say $T < 0.5$ seconds. However, the ranges of parameters in most of these studies were such that only relatively strong systems in the medium to long period range were investigated, with the result that computed ductilities were relatively low. In addition to the elastic period, a useful index of the potential importance of 2D response is the 1D ductility as proposed in [1].

The factor of overriding importance is the effect of gravity. If the 1D ductility is large enough so that 2D response is significant, the gravity effect simply further magnifies the difference. This quite consistent trend is shown in Fig. 2. The gravity parameter P/KL is the fraction of elastic critical buckling load, K is the elastic stiffness, L the column height. For static loading, the elastoplastic system becomes unstable at a ductility of KL/P. When gravity effects are included, the amount of hardening in the system and the duration of strong motion are of great importance [2, 10]. If the slope of the resistance curve after initial yield is greater than the slope P/L, then instability will not occur. Reinforced concrete columns often have a equivalent hardening slope of 5 to 10 percent initially which seems to indicate that there is no realistic possibility of instability since axial loads are not usually large enough to exceed these values. However, as deformation increases the equivalent hardening slope reduces drastically so that in fact instability is possible. The duration of strong motion is of importance since for an earthquake of a given intensity, the greater the duration, the greater the chance of exceeding the response level necessary to cause instability.

Fig. 3 shows responses to El Centro (1940) obtained by Takizawa [13] (no viscous damping) for an elastoplastic resistance curve (labelled "ductile skeleton") and a multilinear deteriorating resistance curve (labelled "deteriorating skeleton"). The system elastic period $T = 0.3$ sec and $P/KL = .006$, both based on the initial linear portion of the resistance curve. Vertical arrows indicate that instability occurred at the next higher earthquake intensity level. For the deteriorating resistance, care must be taken in relating these results to those of other studies since the multilinear resistance curve changes slopes before reaching the "yield level," U_Y. Despite this, the same general trends are apparent.

Plasticity theory was used in the investigations reported in [5-8, 11-13] to extend the uniaxial resistance curves to two dimensions. This is a reasonable approach, and is computationally efficient enough to permit study of a wide range of cases, which is essential if a general understanding of the phenomenon is sought. However, when a deteriorating model is

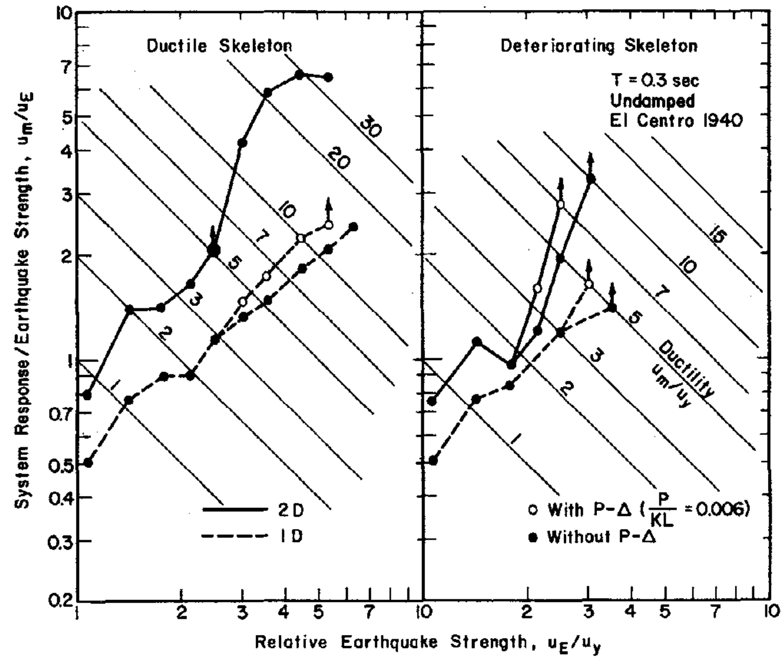

Fig. 3 Effect of Resistance Curve Shape

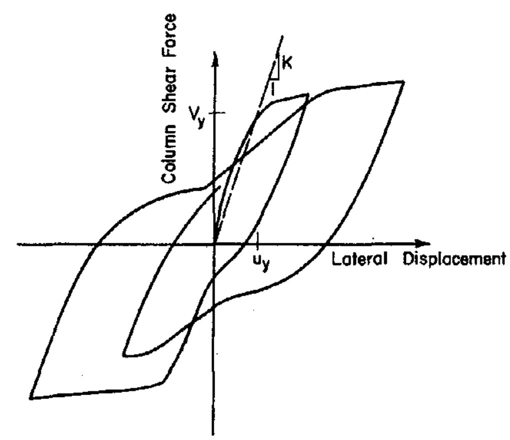

Fig. 4 Shear-Displacement Relation in R/C System

used [13], the plasticity formulation becomes much more complicated. Aktan [1] used a realistic model for reinforced concrete, based on a discretization of the column cross section. Although computational expense prevented a thorough parameter study, more drastic changes in resistance properties and greater sensitivity to 2D motion for T > 0.5 sec were observed than for elastoplastic systems. The excitation levels were rather high in some of the cases, and a need for further investigation was indicated.

In a recently completed study [9], a computationally efficient model has been developed for reinforced concrete columns, which allows a varying axial load. This model is more realistic than that in [1], for several reasons. First, an improved model for the cyclic behavior of the steel is used. Secondly, a more realistic variation of curvature along the column length is assumed. A typical column shear-displacement relation is shown in Fig. 4. The column, whose dimensions and material properties are given in [1], is a typical interior column from the Olive View Medical Center. The axial load on the column is 750 kips which is near the balanced load. The uniaxial yield shear = 131 kips, yield displacement = 0.93 inches and (secant) stiffness = 141 kips/inch. The gravity parameter P/KL = .032 which seems high in the context of elastoplastic systems. However the average hardening slope of the shear-displacement curve up to a ductility of 4 is about 10 per cent, and up to a ductility of 8 is about 7 per cent. The same column properties were used for all response calculations. The column top mass was adjusted to give the desired elastic period, and the earthquakes were then scaled to give varying relative earthquake strengths.

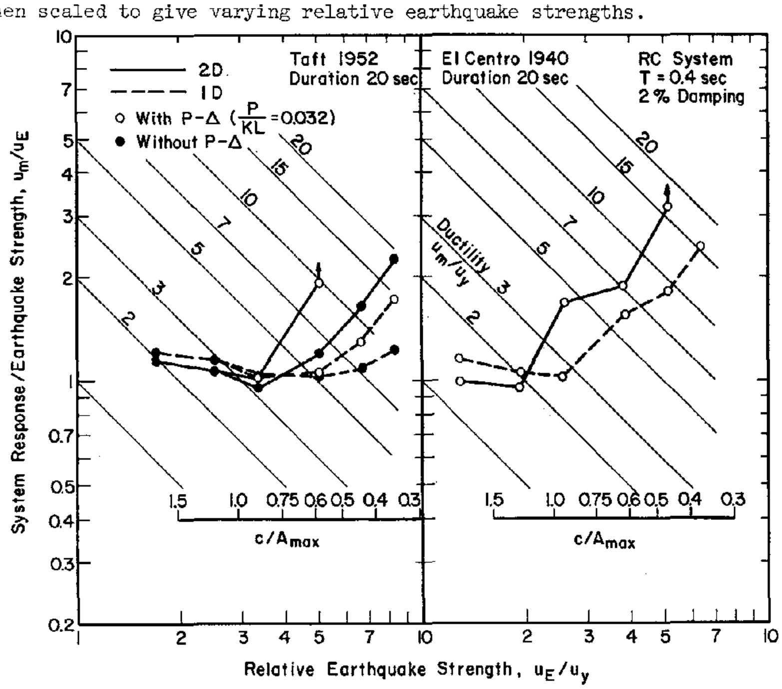

Fig. 5 Response Ductilities in R/C System

Fig. 5 shows results for the reinforced concrete system with T = 0.4 sec (based on the secant stiffness) for Taft (1952) and El Centro (1940). Superimposed on the horizontal scale is a scale of C/A_{max} where C is the base shear coefficient of the system (=yield base shear/weight) and A_{max} is the peak acceleration (in units of g) of the strongest component of the ground motion. The general effects of 2D motion shown are consistent with the elastoplastic models shown previously. The usefulness of 1D ductility [1] as an index for 2D response in reinforced concrete systems is illustrated by the results shown in Fig. 5. When 1D ductility is about 2 or less, the 2D effect is insignificant. This supports the conclusion of [1]. 2D ductilities of elastoplastic systems (Figs. 1, 3) do not seem to show as striking a correlation with 1D ductilities. Nevertheless 1D ductility is a useful index for elastoplastic systems as well. The effects of the gravity load are very consistent, so that it can be inferred when difficulties are likely to occur by an examination of response with the P-delta effect neglected.

Fig. 6 shows a comparison of reinforced concrete systems with elastic periods of 0.4 sec and 1.6 sec subjected to Taft (1952). Clearly, the 2D effect is not confined to short period systems. Elastoplastic systems show the same effect though perhaps not quite as pronounced, when the relative system strength is low enough to produce response ductilities greater than 4 or 5.

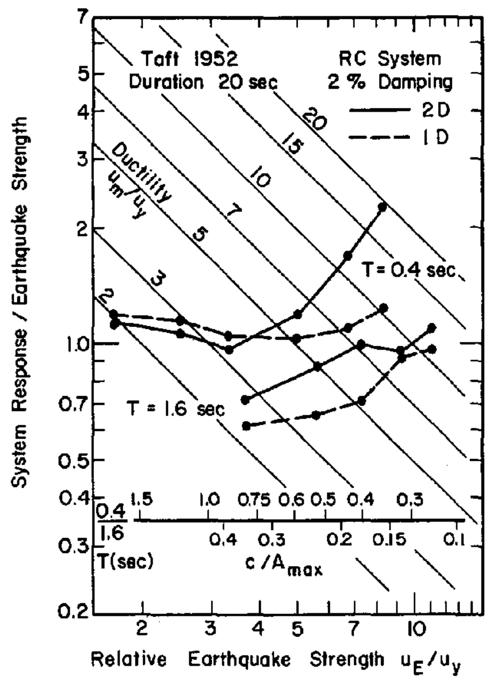

Fig. 6 Effect of Elastic Period on Response Ductilities in R/C System

DISCUSSION

In summary, 2D excitation of single mass systems produces a greater period shift, which in turn can lead to larger displacement response, depending to some extent on the initial system period. Gravity loads acting through the increased lateral displacements may cause collapse. Although details of input motion and shape of hysteresis curve play a role, they do not appear to decisively influence the general trends. The combined effect of correlation of the orthogonal components of response and of inelastic interaction generally appears to increase with relative strength of the excitation. 2D ductilities about twice as large as 1D ductilities are typical at 1D ductilities of about 5 or more.

Since the effect of gravity load is consistent, an examination of responses without the P-delta effect is sufficient to indicate possible problems. Two criteria are useful for this purpose: 1D ductility and system period. The most important indicator is the 1D ductility calculated from a one-dimensional inelastic response analysis. If the system strength is sufficient to restrict the 1D ductility to about 2, no difficulties should occur. In conjunction with this, however, the system period should be taken into account, since the consequences of a slight underdesign are more serious for short period (stiff) systems than for long period (soft) systems.

Although the results presented here are for single mass systems they suggest that frames resisting seismic loads in both horizontal directions should be designed so that column deformations do not substantially exceed "yield." An important factor not accounted for by response studies of single mass systems is the distribution of inelastic deformation between girders and columns in space frames. 2D motion alters the distribution from that resulting from 1D motion, since the columns yield sooner in 2D motion. The few results available for multistory structures [6] indicate that 2D motion increases column response ductility and decreases girder response ductility as expected. Preliminary studies have been made of axial load variations due to overturning moment [9]. While a varying axial load does produce large changes in the restoring force characteristics of a single column, when these characteristics are averaged over several columns in a story, the effect on the total resistance curve for the story appears to be slight. The influence of ground motion characteristics should be more thoroughly explored. Besides duration and general intensity level of the excitation, the relative strength of the two components is important. Important initial studies [3, 12] have been reported, but extensive work remains to be done along these lines.

ACKNOWLEDGMENT

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED
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University of California, Berkeley, July 11-15, 1977

AN OVERVIEW OF THE STATE-OF-THE-PRACTICE OF THE USAGE OF COMPUTER PROGRAMS

by

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INTRODUCTION

The advent of the digital computer has resulted in an extensive reorientation of structural theory from hand calculations to computer methods. Matrix formulations, numerical analysis techniques, and computer programming now play vital roles in the analysis of structures. In the past twenty years, the state-of-the-art has advanced through the development of the first static analysis computer programs for the analysis of frames and trusses to the present programs that incorporate sophisticated finite elements into linear or nonlinear, and static or dynamic analysis. The first programs were restricted to relative small structures with a limited number of joints. In the present technology, the availability of large, fast computers has increased the size and complexity of the analytical model to be limited only by the engineer's imagination and available financing. This leads to the general impression that today's engineer, armed with the appropriate programs, can develop highly refined analytical models of structures subjected to various loading conditions and perform the analysis quickly, easily, and with a minimum of errors.

What is the state-of-the-art of currently used computer programs for the analysis of reinforced concrete structures? Has the "ideal" program been developed? The answers to these two questions are complex and should not fairly be answered by a single author. The objective of this paper is to present the requirements of the engineer using computer programs, the components of the computer program, and a survey of available computer programs. The answers to the two questions will hopefully be better understood even though no answers will be given.

COMPUTER USAGE IN THE DESIGN OFFICE

The practicing engineer must determine the static and dynamic forces on the building, distribute these forces in the structural system, and detail a final design. He must provide a design that meets all the requirements for service loads and also meets less defined requirements for moderate to large earthquake ground motions. The present trend is to rely more and more on sophisticated computer analyses in determining both the dynamic characteristics of the structure and thus the seismic forces, and the internal distribution of these forces to the structural elements whether they be frame members or shear walls. The desired goal of the more complex computer analyses is to produce more accurate solutions, better and safer designs, and thus provide an improved service to the client, while hopefully reducing the engineering effort (cost).

The general usage of computer programs in design offices has fallen short of the above goal. Except for the larger design offices, computer analyses have been undertaken only to satisfy building departments who request computer analyses of complex structures. A reluctance to incorporate the computer into the design sequence of operation can be related to several items:

1. Lack of availability of computers that can be easily accessed without a large expenditure of engineering time.
2. Lack of understanding of the computer programs, their usage, and their results. Manuals for computer programs tend to be complex.
3. Lack of confidence that the computer model is a "true" representation of the structural system.

Larger design offices have overcome the above problems by having in-house computer systems and by developing computer programs or modifying existing programs to meet the requirements of their engineers.

The computer is a design tool which, if properly used, should improve the accuracy of the structural design process, save engineering effort, and hopefully improve the structural system. The role of the computer in the design process should be clarified. The following sequence of steps could be reviewed as the possible steps in a design office:

1. Estimate structural configuration, member sizes, vertical loading, and total mass.
2. Estimate dynamic properties including the fundamental period of vibration.
3. Generate seismic (code) equivalent static forces.
4. Define analytical model and choose a computer program.
5. Perform a <u>computer</u> analysis to determine deflections, stresses, and fundamental period.
6. Evaluate results, adjust dynamic properties, revise structural system and loadings, if necessary, and reanalyze.
7. Perform a <u>computer</u> dynamic analysis using Response Spectrum approach. Determine deflections, forces, and ductility demands for each mode of vibration, and combine the modal responses in a rational manner.
8. Modify structure and reanalyze, if necessary.
9. Perform a <u>computer</u> time-history linear dynamic analysis. Calculate deflections, forces, and ductility demands.
10. Perform a <u>computer</u> time-history, non-linear dynamic analysis. Determine structural stability and degree of non-linear deformations for extreme seismic loadings.

All structures do not warrent all ten steps of analysis. The engineer may choose to only perform an equivalant static lateral analysis if he feels the behavior of the structure is well understood and code force levels are adequate. The response spectrum analysis would provide the engineer with a better understanding of force levels and deflections during projected earthquake levels. Linear and non-linear time-history analysis are usually only performed on very complex structures where the interaction of the modes of vibration and the non-linear response cannot be rationalized by the response spectrum approach.

The development of the analytical model and the choice of a computer program is a task involving many decisions by the engineer. The following partial list must be answered during the development of the analytical model:

1. Should a two or three dimensional model be used?
2. If a three dimensional model is chosen, should it be a true three dimensional model or an assemblage of two-dimensional frame models in a psuedo-three-dimensional model?
3. If a series of two dimensional anaylses are used how should biaxial bending be handled for columns at intersecting frames?
4. How should vertical accelerations be handled?
5. Should the joint zone in the frame be considered a rigid zone (finite joint size)?
6. Should the diaphragms be considered rigid or should they be modeled as flexible membranes?
7. Should the foundation flexibility be considered in the analysis?
8. How should non-structural elements be incorporated into the model?
9. Should the calculated periods of vibration be "adjusted" to be more realistic?
10. Should cracked or uncracked concrete sections be used?
11. Should shear walls be modeled as deep beams or should finite elements be used?
12. How should complex shaped shear walls with many openings be handled such as found in elevator and stair cores?
13. Should buildings with perimeter frames be considered as "tube" structures?

To many engineers the above questions are viewed as arbitrary and approximate assumptions that are used to develop a refined computer model to produce "exact" results. After the engineer has successfully answered the above questions, he must choose an available computer program. The choice is between general purpose programs and programs specifically taylored for the geometry and loadings of typical multistory buildings. Hopefully, a computer program can be found that can represent the engineer's analytical model.

The data for the analytical model and loadings must be generated and checked for errors. Some programs have options to help the engineer check the data. These range from preprocessing programs which only check the data to very sophisticated graphical methods which display the geometry of the structure. When the data is verified, the program and the data are processed and results obtained.

The engineer is now faced with the problem of reviewing the output and accepting the results. The following are some of the questions that require answering:

1. Is the data correct? All errors in the data must be found.
2. Does the computer program correctly model the structure? Deflections, forces, and stresses calculated for the model must be rational for the "real" structure.
3. Is the solution numerically stable? Computer programs rely on

numerical analysis techniques and not on exact mathematics, thus all results must be considered only approximate.

4. Is the solution sensitive to small errors in input data? Small changes in geometry, loads, or damping may have large effect in the solution.
5. Do the calculated forces follow logical paths?

CONSIDERATIONS IN SELECTION OF COMPUTER PROGRAM

After the engineer has formulated his problem (structural system and loading parameters), the evaluation of available computer programs must be researched. The following topics may be included in this evaluation process:

Analytical model Each computer program provides the engineer with the building blocks to create his analytical model. These building blocks may be truss elements, beam elements, and a variety of types of two and three dimensional finite elements. These elements vary from program to program in their representation of the axial, bending, shearing, and torsional deformations which they must mathematically model. Special representations of rigid zones may also be included such as finite (rigid) joint size and rigid diaphragms. The ease at which the building blocks are assembled is related to the versitility of the program in providing enough tools, and to the special purpose features of the program which are taylored to specific categories of structures. The ability of the computer model to represent the "real" structure is beyond the scope of this paper, but it should be realized that the engineer is responsible for the results based on his conceptual model and the final computer model.

Program size The size of the program is related to the maximum size of the analytical model, usually measured as the number of degrees of freedom. A program must be large enough to handle the engineer's problem. Some programs are almost limitless in size (this may not impress the engineer who's problem is usually a two-story, two-bay frame).

Accuracy By the very nature of computers, all results must be viewed as approximate. Both the analytical model and the solution technique have inherent errors. The theoretical differences between the "real" structure and the analytical model exist and their effects on the solution must be understood. The solution techniques, whether they are for static analysis or dynamic anaylsis, are based on approximate numerical analysis techniques not on "exact" mathematics. An understanding of the nature of the numerical errors is necessary in order to understand the results.

Efficiency The efficiency of the computer program is an important parameter in determening the amount of computer time and thus money that is required for the solution of a given problem. There are many numerical analysis techniques for the solution of static or dynamic and linear or non-linear analysis. The efficiency of these techniques is a function of the size of problem and the type and accuracy of results desired. The choice of the technique will significantly affec the computer time required for the analysis.

Input/Output The engineer has to communicate with the computer in order to describe his analytical model and loading conditions and in order to understand the volume of results created by the program. Many programs favor ease of solution by the computer over ease of description of the model by the engineer. The computer output must present all the results required by the engineer and must also satisfy the engineer that the data has been correctly entered, the analytical model restrictions of the program have not been voilated, and numerical stability and thus accuracy exists.

AVAILABLE COMPUTER PROGRAMS

Various general purpose structural analysis programs are available to the engineer through three sources:

1. Universities- Programs developed under grants from different private and government organizations. These programs are available to the public.
2. Private Companies- Programs developed and maintained by private companies. These programs are proprietary and may be used by the public.
3. Consultants- Programs developed and maintianed by private consultants. These programs are proprietary and are used as a part of the consultant's services.

Some of the more well known programs are described in the following text.

Computer Programs Developed at Universities

DRAIN-2D...INELASTIC DYNAMIC RESPONSE OF PLANE STRUCTURES

The program determines the dynamic response of inelastic two-dimensional structures of arbitrary configuration resulting from earthquake-type ground motions. Independent horizontal and vertical excitation may be specified, but out-of-phrase support motions cannot be considered. Static loads may be applied to the structure prior to the application of the dynamic loading, but behavior under static load may be inelastic.

The structure may be composed of elements of a variety of types, each having a different behavior pattern and yielding characteristics. Five different element types have been incorporated into this version of the program, namely, (1) truss, (2) beam-column, (3) shear (infill) panel, (4) semi-rigid connection, and (5) degrading stiffness R/C beam. The program is structured to permit new elements to be added with a relatively small amount of coding effort.

ETABS...EXTENDED THREE-DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS

The program is designed to perform linear structural analysis of frame and shear wall buildings usbjected to both static and earthquake loading-. The building is idealized by a system of independent frame and shear wall elements interconnected by floor diaphragms which are rigid in their own plane. Frame and shear wall elements of arbitrary plan may be specified, within which full kinematic compatibility is enforced. Bending, axial, and shearing deformations are included within each column. Beams, girders, and vertical diagonal braces may be non-prismatic, and bending and shearing deformations are included. Special panel elements allow discontinuous shear walls to be modeled. Finite column and beam widths are included in the formulation. Nonsymmetric, non-rectangular buildings that have frames and shear walls located arbitrarily in plan can be considered. Axial deformations of common column lines of different frames are treated as uncoupled by the program.

Three independent vertical and two lateral static loading conditions are possible. The static loads may be combined with a lateral earthquake input that is specified either as an acceleration spectrum response or as a ground acceleration record. Three dimensional mode shapes and frequencies are evaluated.

LUSH2...COMPLEX RESPONSE ANALYSIS OF SOIL-STRUCTURE SYSTEMS BY THE FINITE ELEMENT METHOD

The program finds the complete response of a plane finite element model representing a soil-structure system. The program differs from more conventional finite element programs in that it, in an appropriate manner, takes into account the strong nonlinear effects which occur in soil masses subjected to strong earthquake motions. This is achieved by a combination of an equivalent linear method and the complex response method. The latter method makes it possible to work with different damping in each element and to consider higher frequencies than most other methods of dynamic analysis.

NONSAP...A STRUCTURAL ANALYSIS PROGRAM FOR STATIC DYNAMIC RESPONSE OF NONLINEAR SYSTEMS

This program is a finite element structural analysis program for the static and dynamic response of nonlinear systems. The system response is calculated using an incremental solution of the equations of equilibrium with the Wilson or Newmark time integration scheme. The nonlinearities may be due to large displacements, large strains, and nonlinear material behavior.

SAP IV...A STRUCTURAL ANALYSIS PROGRAM FOR STATIC AND DYNAMIC RESPONSE OF LINEAR SYSTEMS

This program is a finite element structural analysis program for the static and dynamic response of linear three-dimesnional systems. The program is written to analyze structures which are idealized by combinations of three-dimensional truss, three-dimensional beam, and various finite elements. In a dynamic analysis the options are (1) frequency calculation only, (2) frequency calculations followed by response history analysis, (3) frequency calculations followed by response spectrum analysis, and (4) response history analysis using step-by-step direct integration.

Input data consists of the global coordinated and degrees of freedom of the system nodal points, the definition of the structural elements used, the analysis to be performed (static or dynamic), the description of the system loads and the required output. Data generation is available.

TABS, XTABS, TAB 77... THREE-DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS

These programs are designed to perform linear structural analysis of frame and shear wall buildings subjected to both static and earthquake loadings. The building is idealized by a system of independent frame and shear wall elements interconnected by floor diaphragms which are rigid in their own plane. Bending, axial, and shearing deformations are included within each column. Beams, girders and vertical diagonal braces may be nonprismatic, and bending and shearing deformations are included. Special panel elements allow discontinuous shear walls to be modeled. Finite column and beam widths are included in the formulation. Nonsymmetric nonrectangular buildings that have frames and shear walls located arbitrarily in plan can be considered. Axial deformations of common column lines of different frames are treated as uncoupled by the program.

Three independent vertical and two lateral static loading conditions are possible. The static loads may be combined with a lateral earthquake input that is specified either as an acceleration spectrum response or as a ground acceleration record. Three dimensional mode shapes and frequencies are evaluated.

ULARC... SMALL DISPLACEMENTS ELASTO-PLASTIC ANALYSIS OF PLANE FRAMES

The program computes the node displacements, member forces, support reactions, plastic hinge rotations, and rigid-plastic collapse loads for plane frames of arbitrary shape subjected to static joint loads and support settlements. The program is applicable to low-rise frames of steel or reinforced concrete. Large displacement (P-Δ) effects are ignored. Nonproportional loading, including reversed loading, is permitted. The members may be of nonuniform stiffness and strength.

Commercially Available Programs

ANSYS

Large scale general purpose structural analysis program which can perform static and dynamic structural analysis and heat transfer analysis for both linear and nonlinear problems. Provides extensive plotting capabilities. Capacity approximately 2500 nodes for three-dimensional problems. Developed by Swanson Analysis Sytems, Incorporated.

EAC/EASE AND EAC/EASE 2

Elastic Analysis for Structural Engineering. These programs provide static strctural analyses of linear, three-dimensional systems. These are subjected to sets of arbitrarily prescribed external and thermal loads and displacement boundary conditions. Developed by Engineering Analysis Corporation.

MARK

A linear and nonlinear general-purpose finite element structural analysis program with heat transfer analyses capabilities. Pre- and post- processor plotting options are available. The program is developed and maintained by Marc Analysis Corporation.

NASTRAN

Finite element program which performs a wide range of static and dynamic structural analyses and heat transfer analyses. Several geometry generation options and plotting features are available. Developed under NASA contracts. Oriented more toward aerospace industry applications.

STRUDL AND STRUDL DYNAL

Has static, dynamic, plastic, creep and geometric non-linearity analysis options. Finite element types are restricted to beam, plate and shell elements. Capacity depends on core available and machine size. Oriented toward civil engineering applications. Developed by Civil Engineering Department, M. I. T..

STARDYNE

Performs static and dynamic structural analysis of complex elastic structures. Automatic band minimization, plotting optional. Developed by Mechanics Research Institute.

SPACE

Structural Preporgrammed Analysis Capabilities for Engineers. This program is a large capacity computer program for the static linear elastic analysis of two or three-dimensional structures which may be treated as assemblages of line members and two-dimensional thin plate and thin shell finite elements. Oriented toward civil engineer applications. Developed and maintained by Digital Analysis Consultants, Incorporated.

CONCLUSIONS

Improved numerical analysis techniques, improvements or enhancements to the theoretical modeling capabilities, and bigger and faster computers have and continue to encourage the development of more sophisticated computer programs. These programs will play a vital role in the understanding of reinforced concrete structures and their performance in both the linear and nonlinear stress ranges.

The design engineer may not feel that this rapid advancement in complexity and sophistication of analysis techniques is of direct benefit to him. He does not want to treat each new design as a research project in the understanding of the performance of reinforced concrete structures. What he wants is a simple computer model that can be easily applied to his structure as part of the design process. He desires a minimum of input, a minimum of complexity, and a minimum of cost in both his time and computer usage. A new generation of computers is now available to the engineering design offices. These small, mini-desk-top computers offer the engineer an economical in-house computer system. Simple two-dimensional frame and shear wall programs could be developed for this class of computers. Interactive design options can easily be made part of these programs, such that a designer could analyze, modify and reanalyze a structure during one operation with the computer.

COMPUTER PROGRAMS FOR ANALYSIS OF SEISMIC RESPONSE OF REINFORCED CONCRETE BUILDINGS

by

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INTRODUCTION

Structural analysis provides us with a means of predicting structural behavior. From an academic point of view, it is desirable to predict this behavior precisely; from a practical point of view it is necessary to obtain only sufficient accuracy to ensure a sound design. As structures become more complex, as materials are stressed closer to full capacity, and as regulatory agencies call increasingly for proof that designs are safe, the practioner and the academician move closer together.

In the design of any civil engineering structure, it is necessary to consider serviceability under probable working loads and safety under possible overloads. For structures subjected to typical gravity and wind loads, computations of displacements and deformations at working loads provide sufficient information for checking serviceability, and computations of ultimate strength provide sufficient information for checking safety. For structuressubjected to seismic loads, however, the requirements for serviceability and safety may become substantially more complex.

A particularly important safety consideration is that a strong earthquake may induce structural deformationswell beyond those corresponding to the ultimate static strength, and it is essential that the structure maintain adequate strength under these deformations. Hence, ductility becomes equally as important as strength. An important serviceability requirement for essential structures such as hospitals is that the structural, mechanical and electrical systems must continue to function after a strong earthquake, in which the structural deformations will be substantially larger than those under working loads. In an office building, damage to non-structural components and repairability of the structure are serviceability considerations of considerable economic importance.

It does not necessarily follow that highly sophisticated structural analyses are essential for the design of safe, serviceable structures in seismically active areas. It can not be over-emphasized that sophistication in structural concept, detailing, and construction are of much greater importance than sophistication in analysis. Nevertheless, the designer of an earthquake resistant structure must pay much greater attention to the distribution of forces and deformations in the structure than the designer of a structure in a non-seismic area, and therefore has a much greater need for sophisticated analyses.

In this paper, the state-of-the-art in computer analysis of seismic response is briefly assessed, notable improvements brought about by recent research are reviewed, and recommendations for future research and development are made. The paper is intended as a basis for discussion, not as a definitive statement, so that debate and disagreement are encouraged and welcomed.

STATE-OF-THE-ART: SEISMIC RESPONSE ANALYSIS FOR USE IN DESIGN

A detailed review and explanation of advanced state-of-the-art analysis and design procedures is given in the Applied Technology Council ATC-2 report [1]. However, these procedures are more sophisticated than those commonly used in design.

Reinforced concrete structures are typically very complex geometrically, and are made of materials with complex material properties. Design costs are only a small proportion of the structure cost, so that analysis expenses must be kept low. Sound concepts, details and construction are at least as important as sound analysis. From a designer's viewpoint, the aim of analysis is not to predict the structural behavior precisely, but merely to provide sufficient information to ensure a sound design. These factors all have substantial influence on the state-of-the-art, which can be summarized as follows:

(1) "First order" linear elastic analysis is almost always used ("second order" analysis would include the P-Δ effect).

(2) Structures are typically idealized as assemblages of one-dimensional beam type structural elements. Simple two-dimensional surface elements may be used for walls, panels or, less often, floors. It is common to assume rigid joint regions and rigid floor diaphragms.

(3) Hand computation procedures are still used to a significant extent. However, computer analyses are more common. Computer analyses are more accurate than hand computations, primarily because more refined assumptions and structural idealizations can be used. Computer analyses are not necessarily cheaper, but they yield more information and are more reliable.

(4) Static analyses (for equivalent static design loads) are most often used. Dynamic analyses are used if required by a regulatory agency or if the structure is of unusual configuration. Dynamic analyses are usually of response spectrum type, time history analyses being carried out only in special cases.

(5) Analysis costs for building design are small compared with those for nuclear power plants, aero-space structures and offshore structures. There are few economic incentives for developing powerful, user-oriented computer programs for building analysis and design.

(6) Most computer analyses are carried out using the special purpose TABS program, or its derivatives or similar programs. General purpose programs such as SAP, STRUDL, EASE and STARDYNE also receive significant use. All of these programs are based on the direct stiffness method of structural analysis, and are of a straight-

forward batch processing type. Larger design firms typically perform analyses in-house. Smaller firms are more likely to employ consultants specializing in computer analysis. Many practitioners are still not familiar with computer analysis capabilities, especially for dynamics, but the level of sophistication is increasing.

(7) Three-dimensional idealizations of buildings are common, although two-dimensional idealizations are used where possible. Torsional motions of buildings are often important, and are accounted for by three-dimensional idealizations.

(8) Soil-structure interaction effects are usually ignored, the structure being assumed to be rigidly supported at the foundation level. If foundation flexibility effects are considered, simple spring idealizations are used.

(9) The available analysis programs essentially perform only structural analysis computations, and do not perform code checking calculations on the structural members. Code checking calculations are most often performed by hand (or, increasingly, by desk-top computer) using the printed results from computer analyses.

(10) The available analysis programs typically do not contain many user-oriented features of value for building analysts, so that their use tends to be a specialized task requiring skill and experience. Such user-oriented features would include detailed yet simple documentation; problem-oriented input; automatic error checking; and problem-oriented output.

(11) A number of programs exist which can perform inelastic analyses of seismic response, and hence can produce estimates of the ductility demands on structural members. Inelastic analyses are widely used in research studies, but computational costs are high and there have been few applications in practical design.

In summary, at the time of writing the analysis of buildings for seismic effects is still a fairly specialized task, especially when a dynamic analysis is required. The results of analyses with currently used computer programs consist of structure displacements and member moments and forces, based on first order linear elastic analysis with simple one- and two-dimensional idealizations for structural members. The available programs are sufficiently useful that there are no analysis weaknesses which critically affect the design process, but there is no available program which is really well suited for design office use. Particular weaknesses include lack of problem-oriented input and output, lack of automatic checking for actual or potential user errors, and lack of design checking options.

Because real structures will not behave elastically in a strong earthquake, an elastic analysis will not predict the seismic response precisely, and the suitability of elastic analysis procedures must be questioned. Elastic analyses are used because inelastic analyses are very costly, and because the available inelastic analysis programs have limited capabilities. The assumption implicit in the use of elastic analysis is that if the design loads are appropriately chosen, if the members of the structure are proportioned to resist the computed elastic forces, and if the structure is detailed to develop adequate ductility, then the structural serviceability and safety requirements will be met.

PROGRESS THROUGH RECENT RESEARCH AND DEVELOPMENT

Research and development in computer analysis of structures has led to major improvements in structural idealization techniques, the efficiency of computation, and the useability of computer programs. Notable developments have been as follows.

Direct Stiffness Concepts

The direct stiffness method is a special case of the displacement method of structural analysis, and is a generalization of the slope deflection method. The direct stiffness method is ideally suited for computer application, and has emerged as by far the most powerful analysis technique. Many programs based on this method have been developed and are widely applied in practice.

The physical basis of the method is that the structure is idealized as an assemblage of deformable structural elements (members) connected to a finite number of nodes (joints). In the basic form of the method, the displacements (translational and rotational) of the nodes are the primary unknowns (degrees of freedom) of the problem. The deformations of each element can be expressed in terms of the nodal displacements, and a stiffness matrix for the element can be constructed in terms of the nodal displacements. The stiffness matrix for the complete structure is obtained by direct addition (assembly) of the element stiffnesses. A set of simultaneous equilibrium equations results, which can be solved for the node displacements. The solution is then completed by finding the member forces. The procedure is logically simple and highly automatic, requiring only that the analyst specify the node and member locations, those member properties which govern member stiffness, and the loads or displacements specified on the nodes.

For a general three-dimensional frame, the degrees of freedom consist of three translations and three rotations for each node, which allow for extension, twist and biaxial bending of each beam-type element. In special cases, however, an adequate representation of the structural deformations can be obtained with fewer than six degrees of freedom per node. For example, if a building has horizontal floor diaphragms which may be assumed to be rigid in their own planes, then each diaphragm moves horizontally as a rigid plate, and the horizontal displacements of any nodes connected to the plate are no longer independent degrees of freedom but are "slaved" to the rigid body displacements of the plate. If this type of idealization is used, the total number of degrees of freedom can be greatly reduced, with substantial savings in computational effort. Equally importantly, this type of idealization can substantially improve the numerical conditioning of the problem. The disadvantage of this idealization is simply that the assumption of a rigid diaphragm may not be reasonable, and the analyst must remain aware of the approximations involved.

Provisions for "slaving" of degrees of freedom in this way are included as user options in most general purpose computer programs. In the commonly used program TABS [2] the assumption is an integral part of the program and is always included. This is true also for the ETABS [3], and TABS-77 [4] programs, but some relaxation of the assumption is permitted in the BATS [5] extension of TABS. Other workers, notably Weaver at Stanford, have developed programs for building analysis, but TABS and its derivatives are by far the most widely used.

The principles and procedures involved in selecting degrees of freedom for building analysis are well established theoretically. However, additional development is desirable to determine the simplest and most appropriate idealizations which will allow an analyst either to assume a rigid floor diaphragm or permit it to deform. Existing computer programs are weak in this respect. Existing computer programs are also weak in their provisions for modelling elevator shafts and shear walls which intersect in plan, because of the complex effects of holes and cross section shape on the bending and torsional stiffnesses.

Equation Solving

Direct stiffness analyses of large structures require that large numbers of simultaneous equations be solved. In order to reduce computer costs, a great deal of research and development has been carried out on the numerical operations involved in equation solving, with the result that remarkable reductions have been made in solution times. The research has concentrated on reducing the number of multiplication and addition operations for solving equations by elimination methods (Gauss, Cholesky and Crout procedures), by eliminating unnecessary operations and minimizing storage requirements. Descriptions of procedures and lists of references may be found in [6], [7] and [8]. These procedures are all based on a two-stage procedure in which the equations are first fully assembled and then solved. By contrast, the "wave front" procedure, in which the assembly and reduction proceed simultaneously, is favored by some workers [9]. If equally efficient coding techniques are used, the direct reduction and wavefront techniques require essentially the same solution times.

In the past decade, order-of-magnitude improvements have been achieved in equation solving effort. Current equation solving algorithms are near optimal, so that further major improvements are unlikely. Nevertheless, significant improvements are still possible in equation solving for substructuring and nonlinear applications. Revisions of the algorithms to provide optimal efficiency for new generations of computers may also be necessary

Eigenvalue Routines

Many different techniques have been explored and developed for extracting the eigenvalues and eigenvectors for large matrices, in order to obtain natural mode shapes and frequencies. A review of available techniques can be found in [10] and [11]. A variety of techniques, with different capacities and computational speeds, are incorporated into the computer programs in current use. There is no single procedure which can be identified as "best", and most, if not all, procedures for large systems suffer from lack of reliability in some situations, so that there is still substantial room for improvement.

Substructuring

The substructuring technique is a useful variation of the direct stiffness method. From a physical viewpoint the technique divides the complete structure into separate subassemblages, regarding each subassemblage as a separate structural element ("superelement"). The superelements are then assembled into larger superelements or the complete structure, just like any other element. From a strictly numerical viewpoint the process can be interpreted as assembling and solving the stiffness equations for the structure in separate parts rather than as a single set of equations. The numerical techniques for substructuring have been well developed in recent years, although considerable improvements are still possible. A recent review can be found in [12].

Substructuring has advantages for data preparation, because repeated assemblages need to be specified only once. Computational advantages may also result, because numerical operations on identical substructures need to be carried out only once. The TABS and ETABS programs make use of substructuring, with each discrete frame being a substructure and the substructures being coupled through the floor diaphragms. Some recent general purpose programs [13,14,15] incorporate more general substructuring capabilities, and may be valuable in some cases.

Step-by-Step Dynamics

Seismic analyses for the design of typical buildings are probably best carried out by response spectrum procedures. Time history analyses for specified ground motions are more costly and not necessarily more accurate. The disadvantage of response spectrum procedures is that the responses in different modes must be combined by square-root-of-sum-of-squares and/or absolute sum procedures, and it may be uncertain whether the chosen method of combination is appropriate. A disadvantage of time history analyses is that the computed maximum response may be sensitive to changes in the assumed ground motion. Time history analyses are also substantially more expensive.

The numerical procedures for response spectrum analyses are well established. Step-by-step procedures have received a great deal of attention, and for linear structures procedures which are sound theoretically and efficient computationally have been devised. For a recent review see [16]. For nonlinear systems, however, several questions remain unanswered, and additional study is needed.

Macroelement Idealization

Even with the efficient computer analysis techniques currently available, the computational cost for some structures may be excessive. This is particularly true for large tube-type buildings, in which the columns are closely spaced, with the result that the number of beam and column elements is very large. The problem is compounded because the faces of the tube interact through common columns, so that an isolated frame idealization of TABS type can not be used. A technique for dramatically reducing the number of degrees of freedom and computational cost for such structures has been described in [17]. This technique applies the finite element principles usually used for continua to the faces of a tube-type building, constructing "macroelements" made up of several columns and beams. These macroelements are connected to each other at only a few nodes, so that the number of nodes is much less than for a conventional frame analysis.

Program Structure and Features

In the development of any computer program, decisions must be made on data input procedures, internal program logic, data structure, computational features, and results output mode and format. A "basic"structural analysis program intended for limited use by the program developer is likely to be compact and simple. In contrast, a "production" version of the same program, intended for widespread use in design offices by many people,will require problem-oriented input features, automatic error checking, restart options, results output in final report format, etc. Production programs must also be tested more exhaustively and documented more thoroughly. As a result, the size, complexity and cost are all likely to be much greater for the production version than for the basic version.

The art and science of computer programming for structural analysis has developed a great deal in recent years, with large improvements in simplicity and efficiency at the "basic" program level. However, there are few universally accepted standards or procedures for extending programs beyond this basic level. Developments in data bases, structured programming, tabular decision logic, and portability standards have provided some valuable tools and guidelines, but computer program features still depend primarily on the individual or group developing the program. For example, STRESS and SAP are both successful, widely used programs, but their features from a user's viewpoint are widely different, reflecting the differing philosophies of their developers. Further, program users differ just as much in their preferences.

In the author's opinion this is not cause for alarm, and does not indicate wasteful duplication of effort. Attempts to standardize programming procedures (except, perhaps, with respect to portability and documentation standards) are likely to stifle development and almost certainly to be unsuccessful. On the other hand, carefully prepared guidelines to assist practitioners in the use of programs and preparation of reports can be very valuable. Reference [18] is useful in this respect.

It is also the author's opinion that there is a notable lack of a production type computer program for seismic response analysis. Commercially available programs such as STRUDL, STARDYNE and EASE contain user-oriented features, but because they are general purpose programs they are not as easy to use as would be desirable. The currently available special purpose programs of the TABS type, on the other hand, tend to be merely "basic" programs, with few user-oriented features.

Nonlinear Analysis

Nonlinear analysis problems of many types have received a great deal of research attention in recent years. Inelastic dynamic analyses of simple building-type structures, have been used to demonstrate that ductility demands on well designed structures during strong earthquakes will not be excessive, and have helped to justify the currently accepted procedure in which members are designed for loads well below those which would act if the structure were to remain elastic.

Several general purpose programs, for nonlinear analysis have been developed in recent years, including MARC, ANSYS, NONSAP, ADINA and ANSR. These programs can perform inelastic seismic analyses of buildings, but generally are not convenient to use. Several special purpose programs for building analysis are also available, including INELASTIER [19], the A.C. Martin program [20], the Kamil-Mahin program [21], DRAIN-2D [22] and DRAIN-TABS [23]. These programs are more convenient to use then the general purpose codes, but nevertheless require specialized skills and are costly to run for large practical structures. As a result, there have been few practical applications. Nevertheless, nonlinear analyses are currently indispensable for research investigations. As improvements in computational efficiency are made and as demands for rational proof of structural safety increase, it is inevitable that nonlinear analyses will be used more and more in practical design.

A great deal of research and development remains to be carried out on many aspects of nonlinear response analysis, including improvements in numerical techniques to reduce costs, improvements in the mathematical modelling of inelastic members to increase accuracy, and improvements in program features to simplify program use and produce results which can be used directly by designers.

Summary

The advances from recent research can be summarized as follows.

(1) Highly efficient numerical procedures for linear elastic analysis under static and dynamic loads have been developed. Several improvements are still possible, notably in eigenvalue routines and substructuring.

(2) A number of practical computer programs have been developed, including some intended specifically for building analysis. However, there is still a need for a user-oriented production program for use in earthquake resistant building design.

(3) Promising idealization procedures and computational techniques for nonlinear inelastic analysis have been developed, and a number of usable computer programs are available. However, there is need for additional work on the idealization of inelastic members, on efficient numerical procedures, and on developing user-oriented computer programs.

OUTSTANDING PROBLEMS

Several problems remain to be solved, in the areas of structural idealization for linear elastic analysis, numerical techniques for linear elastic analysis, procedures for nonlinear inelastic analysis, production program development, and correlation of elastic analysis results with actual inelastic behavior.

Idealization for Linear Analysis

(1) Regardless of whether a general three-dimensional idealization or a TABS type idealization is used, problems arise in modelling shear walls which intersect in plan to form wall structures with I, angle, box, etc. cross sections. These wall structures may have complex flexural and torsional properties, especially when pierced by holes, and these properties are not considered rationally in most analyses. There is a need for the most appropriate idealization procedure to be identified. This procedure might use plane stress finite element representation of the walls, thin-walled beam theory [24], or some other procedure.

(2) Floor diaphragms are commonly assumed to be rigid in comparison with the frames or walls which they connect, and this assumption is not always reasonable. There is need for improved idealization procedures to account for floor diaphragm deformations, and for guidelines to indicate when the assumption of a rigid diaphragm may be unreasonable.

(3) Soil-structure interaction effects may be significant for stiff concrete buildings. A practical procedure for accounting for these effects is described in [25], but techniques for incorporating such a procedure into building analysis programs need to be studied.

(4) Research on the effective width of floor slabs acting with beams, the effects of rigidity or flexibility of joint regions, and the effects of shear deformations need to be reviewed, and guidelines prepared on appropriate modelling to ensure that member stiffnesses are represented with reasonable accuracy.

Numerical Techniques for Linear Analysis

(1) There is a need for a review of existing techniques and subroutines for eigenvalue determination, with the aim of identifying the most reliable and efficient procedures.

(2) Substructuring techniques hold promose for reducing both data preparation and computer time, but have not been explored extensively for building analysis. These techniques warrant receiving more attention.

(3) "Macroelement" techniques promise major reductions in computer time for tube type structures, and warrant receiving more attention.

Nonlinear Analysis

Nonlinear analysis is still relatively in its infancy, yet is important if truly rational analyses of structural safety are required. Virtually all aspects of nonlinear analysis warrant additional study, from idealization procedures through numerical techniques to the meaningful presentation of results.

Production Program Development

(1) There is a definite need for a production type computer program for design office use. The most appropriate structural idealization on which to base the program is probably that of ETABS, which permits frames to interact fully through common columns if desired, but also allows the simple TABS idealization of plane frames coupled only through the floor diaphragms. The program should ideally permit accurate idealization of intersecting shear walls, walls with openings, non-rigid diaphragms, non-prismatic members, and soil-structure interaction. The program should have problem-oriented input and output. Consideration should be given to providing options for checking against Code requirements. The program development would require close cooperation with designers, and might most appropriately be carried out by a commercial firm rather than a University.

(2) The Code provisions for checking reinforced concrete members designed for seismic resistance have not been framed with computers in mind, and hence may not be in appropriate forms for incorporation into a design analysis program. Procedures for analyzing the logical structure of Code provisions are well developed, using tabular decision logic and other techniques. There is a need to examine the existing provisions with a view to recasting them, if necessary, in a form more amenable to incorporation into computer programs.

Correlation of Elastic and Inelastic Behavior

It is implicit in current design procedure that if the members of a structure are proportioned to resist the forces computed by linear elastic analyses, then the actual ductility demands on the members during a strong earthquake will not be excessive. With recent advances in inelastic analysis techniques, it is possible to check whether this implied assumption is reasonable. Aspects which might be studied include (1) whether a tall frame designed with the same load factor in all stories actually experiences the same ductility demand in all stories, and (2) by how much the columns of a tall frame should be overdesigned to ensure that weak-beam-strong-column behavior is obtained. Some recent studies, for example [26] and [27], have looked at these aspects, but further investigations are required.

CONCLUSION

This paper has briefly reviewed the state-of-the-art in computer analysis for the design of reinforced concrete buildings, has listed several areas in which significant advances have been made in recent years, and has identified a number of problems which are still outstanding. The most important needs for future work are believed to be (a) development of a production-type building analysis program, (b) continued research in nonlinear analysis techniques, and (c) correlation of the results of elastic and inelastic analyses.

Specific items for study are identified in the attached Draft Recommendations.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

ELASTIC ANALYSIS OF WALLS WITH OPENINGS

by

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Much valuable information is being generated by inelastic cyclic experiments for the earthquake-resistant design of walls. Such research provides insight into the ultimate behavior. However, in practice walls are actually designed on the basis of linearly elastic analysis. When walls are interconnected with other walls and slabs, and, in addition, have openings, an accurate analysis of such structural systems becomes a time consuming task. As an aid to the designer, a special purpose finite element computer program for practical elastic analyses and design of structural walls with substructure option has been developed. This note simply calls attention to the availability of this program titled SUBWALL.[1] The abstract for this program reads as follows:

> "An efficient and refined special purpose finite element computer program is developed for the linear structural analysis and design of complex reinforced concrete walls subjected to arbitrary in-plane static loadings.
>
> A substructuring technique has been implemented along with several practical user's options which contribute to the computational efficiency and economy unavailable in many general purpose computer programs.
>
> Large structural walls with multiple openings, nonplanar coupled walls, and staggered wall beams systems can be analyzed. Openings and offsets in structural walls are represented by special "hole" elements with no structural stiffness.
>
> Preparation of input is simple. Few cards are necessary for a relatively large size structure. Joint coordinates, element connectivities, and boundary conditions are automatically generated.
>
> Output can be requested in terms of tabular printout, element by element printer plots and/or regular plotter plots of displacements, stresses, and reactions. Section forces such as shear, moment, and normal force in connecting beams can be obtained.
>
> Main emphasis of the report is on the practical and economical application of the finite element method rather than the theoretical aspect of the program development. Throughout the report, linearly elastic behavior of the materials is assumed."

The program and instructions for its use are available from the National Information Service Office, Earthquake Engineering, 337 Davis Hall, University of California, Berkeley 94720.

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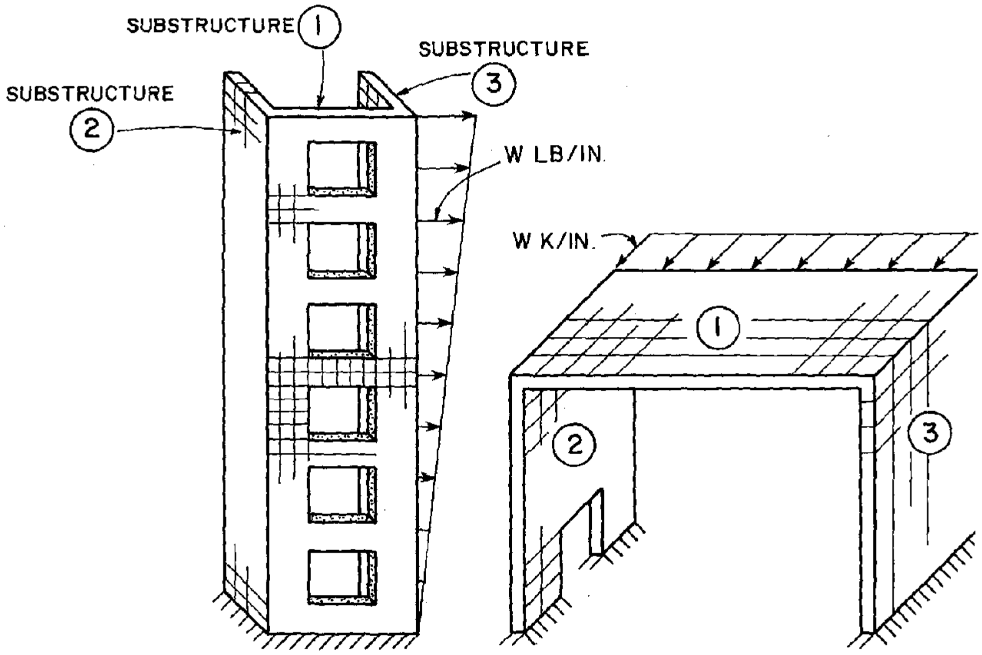

(a) Nonplanar coupled wall with multiple openings

(b) Planar walls with coupling slab and opening

FIGURE: ANALYSIS BY SUBSTRUCTURING OF TYPICAL SYSTEMS

WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

COMPUTER AIDED DESIGN OF EARTHQUAKE RESISTANT REINFORCED CONCRETE BUILDINGS

by

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INTRODUCTION

The majority of reinforced concrete buildings being designed and constructed today are of modest size. They are large and complex enough to require seismic analysis, but not large enough to justify comprehensive detailed dynamic analysis. As a result they are usually designed using a combination of hand and computer aided methods. This paper will describe some of the common computer aided seismic and gravity design techniques available to be used and why they are not used by the engineering community in daily analysis and design of these structures.

LIMITATIONS

First the Engineer's choice to use computer aided design is often limited by a number of interrelated constraints that have prevented his rapid and complete acceptance of computer methods.

Knowledge

Knowledge or the lack of it is one direct limit, whether it is the lack of knowing where to be able to obtain the use of specific software or of how to avail oneself with the access to the use of a computer. Likewise there is a lack of understanding by many Engineers of the degree of assistance obtainable through computer aided design. Sometimes there is fear, other times there is simply a lack of understanding of computer techniques or of the problem solution algorithum which leaves the Engineer unable to accept the responsibility for a computer aided design.

Access

Availability is another limit. If the method of access is time consuming or cumbersome it often is simpler and more economical for the Engineer to continue to use hand methods. This limit is often measured against the scope of the problem solved. That is to say in many cases only large problem solutions can justify the effort required for the results obtained.

Codes

Another limit which is generally more related to design than analysis is the conformance of the resulting computer aided design to published Building Code Standards. Many uses of computer solutions are by passed because in some detail they fail to meet governing local Code and hand adjustment or recomputation is either too time consuming or impossible from the results given.

Economics

Economic considerations also place some severe limits on the application of computer aided design although the availability of significantly improved equipment and software at greatly reduced cost has reduced this barrier. The Engineer must adapt to employ regular and substantial computer usage or he cannot justify the cost of an in-house facility with its attendant support costs. As an alternative, unless the Engineer can find substantial amounts of well documented, useful code current software either on timeshare services or at service bureaus he cannot justify the costs entailed in training for and utilizing these methods either.

Let's examine how Building Design Codes have contributed to this problem and how it is being overcome by some Engineers.

First, codes have been developing and changing very rapidly since 1971. The SEAOC "Recommended Lateral Force Requirements", commonly called the Blue Book, has been substantially changed recently both in analysis of forces and in the design requirements for concrete structures. The Uniform Building Code which is based largely on the Blue Book has similarly changed. New ductile concrete requirements have been developed and are still being developed. For instance the requirements for reinforced concrete design of the 1971 ACI (Appendix A) and of the UBC for seismic design are quite different. ACI Committee 352 has just published a guide for the design of concrete frame joints, a subject which has been largely undefined from a code standpoint.

Titles 17, 21 and 24 of the California State Administrative Code have been rewritten several times in recent years to reflect changing knowledge and to satisfy the mandate of the law which in some cases now requires dynamic design.

The ATC-3 project to produce "Recommended Comprehensive Seismic Design Provisions for Buildings" the latest in suggested design codes has been undergoing considerable discussion as Engineers, scholars and researchers try to codify more realistic design methods based on risk, dynamic response and material yield level resistance.

Program Development

In respect to computer aided design then, the result of all of this has been a delay in the creation of integrated analysis and design programs. A comprehensive computer program requires a firm expression of requirements before it can be written. Costs of rewriting such programs are great and difficult to amortize against short periods of use.

We have instead seen the development of separate analysis and design computer programs and much effort devoted to relating dynamic analysis results back to common static equivalents so design can proceed by more conventional methods for which computer programs and hand methods of design exist.

Dynamic Methods

At present, dynamic design methods have not been codified or even standardized. The Electronic Computation Committee of the Structural Engineers Association of Southern California is currently engaged in publishing "Recommended Guidelines for the Performance and Presentation of Computer Generated Seismic Analysis of Structures" to begin to bring some order and understanding to the methodology applied in typical design.

It should be noted that the ATC-3 study which has researched and collected some of the latest seismic analysis design methodology does not prescribe true dynamic design techniques. It does offer the equivalent of spectral values based on modal period determination from which equivalent static forces can be determined.

Even if true dynamic forces could be set through the use of a standard dynamic approach, including accurate modeling techniques, the Engineer currently has the limitation of what are Code acceptable dynamic resistance values and how can these be combined with statically determined dead and live load forces.

When we remember how many static design rules have been set by material misuse, or abuse, deflection control, ease of approximate solution and similar non-load and non-time related considerations we begin to realize the dilemma the Engineer faces. There are no time factors in static loads. What about peak stresses? What about skip loads, partition load allowances, live load reduction, arbitrary building torsional response allowances, etc.? How should these be combined with say the plate stress of a finite element of the edge of a bearing wall or at a beam column joint? Not to mention the difficulty of determining a model for, say a frame, shear wall structure with semi-flexible diaphragms and an irregular plan layout. Further there is uncertainty in the determination of the stiffness of concrete members. What is the effect of cracked sections on I? If we have difficulty in predicting the deflection of beams under static load, how do we predict the deflection performance of frames under dynamic load?

Equivalent Static Forces

As a result, most design today is accomplished by finding equivalent static forces for seismic action at code level resistance stresses. Computer aided design methods are available for these once the dynamic forces have been reduced to psuedo static forces.

To first obtain dynamic forces a number of computer programs are available such as TABS, SAP, DYNAL, NASTRAN, STARDYNE, etc. The use of these programs is often limited by the reasons described earlier, and particularly by the cost of analyzing complex models. As a result models are often greatly simplified to reduce degrees of freedom. Multistory irregular shear wall structures can easily run into thousands of degrees of freedom for an accurate model and solution of this is not in most design budgets. Often these structures are either redesigned to be more regular so that the static design methods can be substituted or static methods are simply used.

SHEAR WALL BUILDING

Let's examine computer aided design procedure for a simple shear wall building, and see the kinds of programs that could be used.

First the Engineer has the choice of a dynamic program such as TABS which uses a simple model or more complex treatment of finite elements such as a version of SAP. We will assume he can circumvent the limits such as availability, fees, costs, and understanding of the applicability of each to his structure. After simplifying and modeling and having made decisions on ductility, dampening modeling effects, elastic or non-elastic performance, choice of seismic excitation, site effects and soils structure interaction he will obtain some results. If these are time related he must further make some assumptions on what to choose as a static equivalent to proceed with a normal code design to develop material requirements. For instance he could select the story shears effective at the most deflected position of the structure to get an overall equivalent static load.

Generally at this point he will need to expand his simplified model back to the actual elements and ascertain the forces on each part due to seismic and gravity forces to design sizes and reinforcing steel.

Static Design

Let's examine how this static design might be done using reasonably accessable existing computer aided design software.

Note that these same processes could be used in preliminary design by accepting code static forces or some equivalent as a starting point to size a structure.

Shear wall analysis - If we use as an example a simple shear wall structure, the Engineer would go through the following steps to use the computer aided design capability of the "Multi-Story Wind or Earthquake Rotational Analysis" program, one of a group developed by Systems Professional and available on several national timesharing bureaus, and on several makes of mini-computers.

First he lays out his structure in plan, sets the coordinates and dimensions and numbers the walls. Using code sheets provided he furnishes a description of the geometry of the various walls on various floors and establishes through simple indicators how he wishes to assume walls be assembled from piers and spandrels to establish rigidities.

The methodology is the same as his hand methods but the calculations performed are more rigorous than he would normally use.

Once geometry is established he can either describe the weights of portions of the structure or apply previously determined dynamically related static loads at each floor level. If he uses weights, he then sets the ZIK and T_s and T values or in the case of the latter lets the program choose them by code. Last he sets story heights and gross building dimensions. For this nominal amount of information he gets all the rigidities of elements and walls, a period and force determination, an actual or arbitrary Code required application of torsion and all shears and overturning moments for all walls at all levels.

Pier design - He can then proceed to take these forces together with gravity loads to a "Masonry, Block or Concrete Shear Wall and Pier Design" program and determine what kind of reinforcing is required, single or double curtain, or special edge bars based on ultimate moment methods of the 1971 ACI.

Bearing wall design - Next, bearing walls can be designed for perpendicular seismic loading using the "Masonry, Block or Concrete Bearing Wall Design" program, also based on 1971 ACI code.

These programs take minutes to code and process so preliminary results are quickly and easily obtained.

Floor design - Floors can be computer aided designed by a number of programs. Flat plate, waffle slab and two way slab analysis and design programs have been made available by PCA through purchases, or through service bureaus or by timeshare services. These programs will analyze and design for either gravity or for gravity plus seismic joint moment effects as determined from other frame analyses. For some cases full reinforcing layout can be obtained in addition to full shear and moment analysis and steel area design.

Concrete beams can be computer analyzed for all types of live and dead loading and seismic joint moments using either "Prismatic Continuous Beam Design" or a similar non-prismatic version.

Steel areas, stirrups, inflection points etc., are all automatically determined.

Prestress design - Even if the floor is prestressed the slabs, beams and girders can be computer designed using Posten a program developed by Hugh M. O'Neil Company and available nationally by timeshare or service bureau.

Column design - Concrete columns can be designed for biaxial bending plus axial loads using either a single story design program or multiple story load take off and design computer programs.

Again the Engineer furnishes only modest design parameters.

Footing design - Footings whether rectangular, square or tied can also be computer aided designed.

Ductile Requirements

None of the above programs address ductile design requirements though and these will have to be added by hand.

FRAME BUILDING

Let's examine the analysis and design when frames are the seismic resisting element and using computer aided design. Here the Engineer can find more assistance.

Frame Analysis

First he has the choice of either stand alone or integrated programs, some of which will address both static and dynamic analysis using nearly the same input data. In a few cases the programs can be directly linked to design of the concrete members. In other cases the joint moment data must be transferred to floor design programs to automate all of the code provisions for skip loading.

The following programs can be used for either 2 or 3 dimensional frame analysis:

"Tabs" or one of its later versions available from NISEE is frequently used to perform linear structural analysis of frame and shear wall buildings for both static and dynamic loadings. It involves some simplifications of analysis which limits its use on some types of structures.

"Sap" or one of its later versions also available from NISEE can be used to perform a more complex finite element analysis either linear or non-linear for both static and dynamic loads. Costs mount with the number of elements and so usage tends to be restricted to the more important design projects.

The "Strudl-Dynal" programs are portions of the ICES system and available from McAuto. These programs use problem oriented language and have common data bases and include finite element capabilities.

They will perform static and dynamic design and have the added advantage of being linkable to a "Reinforced Concrete Structures" design link for either investigation or design. The Engineer has considerable freedom in problem description and in controlling the members to be designed making the programs useful for preliminary design as well.

"Nastran", "Ease", "Space" and other programs are also available for both static and dynamic design but are generally used only for the more complex analysis problems, usually where finite element capability is desired.

Choice by the Engineer of the above programs is usually made based on availability to him, prior usage knowledge, or cost.

For the more common structures, where elaborate analysis is not justified, the Engineer will more often use only the static design capability of the simpler frame programs, listed above, applying lateral forces separately determined from simple models using a single mass at each floor or just code specified forces.

For concrete structures frame programs like "Strudl" or "SPStress" are advantageous to the Engineer in that they use the free form input of problem-oriented language, a wide choice of loadings and structure features, the ability to factor and combine loadings, the collection of loadings by member, and the ability to obtain moments at segments along the members. These last features can greatly reduce the designers time in selecting members for further design.

Reducing Output

This then touches on the next problem for the Engineer associated with automated design. Namely controlling the volume of output, and the costs associated. While it is possible to describe all skip live loading, dead loading and seismic loading for a building in one huge computer run, in order to seek maximums for design this is rarely done. More often runs are limited to seismic, dead and full live load and these are factored together to produce the ultimate full load combinations required by code for lateral design. The designer can then judge where critical portions of the structure are for design and submit only these to further design.

Floor Design

As previously described the Engineer can turn to individual floor member design programs which examine a continuous beam with columns above and below and superimpose frame joint moments with gravity loads as well as perform the necessary skip load code requirements. The remainder of the structure can be computer aided designed as described for shear wall structures.

SHEAR WALL AND FRAME STRUCTURES

Mixed structures pose some special design problems for the Engineer by Code. If the building is essentially á frame with wall panels introduced for deflection control or building core protection, the walls may be described in most of the frame programs previously described and treated according to the assumptions of the program. For finite element programs wall descriptions can easily be included, but size of run increases rapidly, particularly with non-solid walls.

For essentially shear wall buildings where a frame is introduced to reduce code seismic force requirements, the analysis process is complicated. Here the Engineer must perform a shear wall building analysis as described before, and determine the deflection pattern of this structure. Then, assuming code loads have been used he must increase these deflections by some factor of the code specified K and analyze the frame for these deflection effects. Here "Strudl" or SPStress" can be effectively used for computer aided design and obtain shears and moments to be applied as described before in design procedures.

DATA BASE APPROACH

Recently much effort has gone towards a data base handling of computer aided design.

The ICES package of which "Strudl" and Dynal" are subsets is intended to address this approach. The intention is to allow the Engineer to gradually submit information and request that portions of analysis or design be performed, this way he interfaces and can control and limit results. Costs of learning, equipment, errors and storage limit this process at present, but it is moving. Larger companies with in-house equipment can effectively utilize this process.

The "Genesys" package has a similar approach but due to software efforts the system is somewhat machine independent and is fractionated such that it can be mounted on smaller equipment.

It makes extensive use of standardized tables to allow program and user interface. Substantial effort has gone into developing reinforcing bar bending schedules as final output which can eliminate much drafting time for presentation.

The "Taskmaster" data base system has adopted a more complex data base storage system for building analysis and design and stores information for each member of the building in a relational way. Analysis and design programs can be readily interfaced to this data base, extracting geometry or loading information and performing analysis and design in parts with the results re-entering the data base. The end object is computer graphic production of drawings. A system is operational for production of steel drawings, but much work remains to be done for concrete design.

NEEDS FOR THE PROFESSIONAL

What then are the needs to improve the use and results of computer aided earthquake resistant concrete building design?

First it would seem we must achieve some agreement on the methodology and values for dynamic design. While much of the mathematics of solution and supporting computer programs have been developed in recent years, the modeling techniques and input forces are more often based on opinion or assumption. We have much work to do and data to gather on real structures and real events to be able to formalize these procedures.

Usable values for seismic design involve social decisions as well. While perhaps with great effort and expense all buildings could be designed for the worst event ever experienced, economics do not justify this freedom from risk. Such an event may never happen to a particular building in the life of that building. Predicting site related forces from an event still is not a reality. Predicting the interaction of the soil and the structure is still in the development stage. Code procedures for static load foundation overturning design are poorly specified and little understood.

There is much work to be done in correlating complex non-linear analysis of realistic structures with simpler usable design procedures. While important or vital or dangerous content structures may justify substantial design effort, surely we can develop simpler equivalent design procedures which provide adequate protection to life and property for the majority of structures.

We need simple ways to generate computer data for complex structures. This may have to come from reverse graphics. We cannot cause the Engineer to have to become a computer expert in order to process data through a program. We need much technology transfer of existing capabilities.

While we work hard on difficult theoretical solutions, we still lack much data on the behavior of materials under dynamic and perhaps often non-linear conditions. How do we relate this behavior into reasonable, simple design procedures that can be implemented in an engineering office? The report by ACI-ASCE Committee 352 on beam column joint design is one of many subjects that need this type of study and treatment.

Bar development and tie requirements for ductile frames even as now expressed in codes are extremely complex, perhaps too much so to be readily implemented by the Engineer or to be properly placed by the fabricator. Maybe we need to develop simple splice mechanisms that allow all bars to be continuous.

How much of the slab in beam slab buildings will participate in the frame stiffness? How should likely hinge locations be designed for the best non-linear behavior and to continue to sustain gravity forces also present? What's the effect of long time creep transfer of load to the reinforcing of the columns when a dynamic event occurs?

How should we design bearing wall structures? How do we realistically combine buckling effects in a bearing wall with the varying load of overturning? Do we know how to predict the shear capacity of a complex shear wall with numerous openings? What should be done to control shrinkage cracking in buildings which often places cracks in key elements long prior to a seismic event?

We need to develop realistic live load requirements to be combined with seismic forces. This obviously will have to be related to the system performance of a building as some members may have substantial live load and others none.

We need to relate code design procedures to true material resistance levels such that the Engineer can visualize how a structure is performing. If large deflections are to be anticipated they should be known and provided for in the design.

If brittle materials are going to fail at some point and reduce resistance of the structure we need to have design methods reflect this.

The ATC-3 study previously referred to has attempted to relate recently developed information on risk, seismicity and response of structures to elastic yield level resistance allowing for some inelastic behavior. This type of work will have to be continued and expanded into full codified dynamic procedures before more automated analysis and design procedures can be effectively implemented.

FUTURE OF COMPUTER AIDED DESIGN

It appears there will be a continuing need for the development of more comprehensive and more complex programs to assist the Engineer in his analysis and design. It also appears that to control the cost of design, better approximation methods of analysis are going to be required to allow the simulation of dynamic effects with lesser resources of computer core and storage. Simpler data input methods must be developed. The development of these types of software are beyond the resources of the typical design office and will have to be sponsored through research institutions, and funded by group effort as well as public funds. As the fundamental theory programs become available they have to be verified and tested against real building action and performance. Only after techniques that produce meaningful results have been established can true simplification be developed and tested.

It would seem that design routines will best be developed as modules that treat portions of the total design process so that they can be easily modified and can be used against the data base developed in the analysis of the structure. Current code rules for design are already so complex for full application to a reinforced concrete frame that it requires the unfailing memory of a computer to apply all of the provisions. Ductile provisions, even as they now stand, further complicate these rules. As they are extended they could become even more complicated. We must strive for simplification or few ductile frame structures will be built due to cost.

Spatial relationships will also become primary as we strive for confinement to improve performance. Computer graphics will begin to be most important here to display design results and to search out and prevent conflicts. Ideally if design could be automated to the point of resolving these conflicts and to resolving reinforcing steel with certainty, only schedules and then numerically controlled machine tapes for fabrication would be required. Drawings for concrete outlines can now be easily computer produced to describe the structure.

SUMMARY

There exists a number of computer aided analysis and design programs for the Engineer to use today in proportioning a building for static gravity and lateral loads. He can perform a full preliminary analysis and design on a structure easily and in little time using very modest computer equipment, core and storage.

Until dynamic analysis and design procedures become codified, methodology is going to be confused and difficult to apply in an economical, orderly and uniform manner. Material resistance requires equal attention for compatible results and to incorporate simultaneous gravity requirements.

Data base techniques will probably be necessary to treat the building as a system and to reduce data preparation and handling times and effort.

Much work needs to be done to simplify and standardize methods so cost can be kept under control and additional useful software provided.

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WORKSHOP ON EARTHQUAKE-RESISTANT REINFORCED CONCRETE BUILDING CONSTRUCTION (ERCBC)
University of California, Berkeley, July 11-15, 1977

ON THE USE OF COMPUTERS IN THE SEISMIC-RESISTANT DESIGN OF REINFORCED CONCRETE BUILDINGS

by

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INTRODUCTION

There has been a dramatic increase in the use of digital computers in the seismic-resistant design of reinforced concrete buildings during the past decade. This has been motivated by a number of factors including the ever-present need to design structures that are safe as well as economical, the complexity of the response of structures (particularly those constructed with reinforced concrete) to earthquake ground motions, and the increasing availability of economical, design-oriented computer programs. While there have been some attempts to develop computer programs as direct design aids, the majority of work has been directed towards development of analytical capabilities to predict the linear elastic or nonlinear response of building structures to prescribed static or dynamic actions. The accelerating development of computer programs for these and other design-related purposes portends a great number of possibilities for improving the reliability and efficiency of seismic-resistant design.

For example, the development of reliable and economic analytical methods capable of predicting the nonlinear dynamic response of three-dimensional structural systems (including their interaction with 'nonstructural' elements and the supporting foundation) to realistic seismic excitations would allow designers to obtain quantitative information with which they could rationally evaluate the seismic performance of individual structures. The results of computer-based simulation studies in conjunction with field observations of earthquake damage and experimental research could provide valuable information regarding specific parameters that must be accounted for in the design of different types of structural systems. Such integrated studies could also serve as the basis for formulating simplified design methods for standard or simple types of structures where extensive analysis might not be desirable or economical. In this manner, computer simulations would also provide a useful check on the reliability of building code recommendations.

Another use of computers is to simulate the detailed behavioral characteristics of individual components. Structures are generally designed to dissipate some of the energy input during a severe earthquake through inelastic deformations. The ability to predict analytically the mechanical characteristics of different types of structural components under various types of loading conditions could significantly improve design. Such analytical methods, if integrated with, and corroborated by, experimental data, could be used to: (1) improve the mathematical modeling of structural systems; (2) allow designers to evaluate the ability of specific design details to sustain the internal forces and inelastic deformations that might be developed during future seismic events; and (3) develop standard design details that possess desirable (and predictable) mechanical characteristics. The usefulness of computer simulations ultimately depends on the reliability with which they predict actual behavior. It is consequently necessary to carefully integrate experimental and analytical studies to formulate realistic mathematical models.

Another important use of computers is the automation of standard design practices and in the implementation of design procedures based on more formal mathematical programming and risk analysis theories. Automating all or part of the design process could result in more efficient and economic usage of materials and would free designers from routine and repetitive tasks.

Objectives and Scope

The objectives of this paper are to review and evaluate the current capabilities of computers to aid in the seismic-resistant design of conventional multistory reinforced concrete buildings and, in view of the previous discussions, to suggest areas for future development. To do this, a general framework describing the general design process will be presented, and the application of computers to certain steps in this process will be discussed in some detail. While it is not possible to present a comprehensive evaluation of computer usage within the constraints of a paper, selected references will be discussed to try to put the use of computers in the different aspects of design into perspective. Suggestions for future research and development are offered on the basis of this study.

STRUCTURAL DESIGN PROCESS

Design is the process whereby perceived needs are transformed into physical solutions capable of fulfilling these needs. This generally entails: an iterative procedure (Fig. 1) of synthesizing needs, functional or practical constraints, engineering principles, and other related information into a feasible solution; assessing the value of this solution; and deciding whether the solution is acceptable or must be modified. From this theoretical perspective, the structural design process can be analyzed in terms of a number of disciplines including the broad field of psychology (problem solving), operations research, information control theory, and so on. Becker [1] has reviewed the general philosophy of design and has implimented a computer-based model to aid structural designers to acquire, manipulate, and generate information necessary to identify appropriate design alternatives. While such general computer-implimented design methodologies are powerful tools in understanding the overall design process and in achieving rational solutions to complex multifaceted problems, the focus of this paper concerns the more typical application of computers to the seismic-resistant design of reinforced concrete multistory buildings.

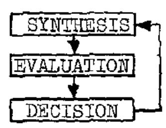

FIGURE 1. BASIC DESIGN PROCESS

The basic seismic-resistant design process considered in this paper concentrates on the seismic aspects of structural design and is schematically illustrated in Fig. 2. This process consists of the 3 components shown in Fig. 1 (synthesis, evaluation, and decision), but the design steps are grouped in different phases for convenience of discussion and to illustrate the potential use of computers. It is important to realize that the actual interdependancy of steps is much more complicated than the unilateral processes generally indicated in this figure. For example, minor changes in the structural system (e.g. increased mass) may effect the gravity loads, dynamic response characteristics, the magnitude and distribution of seismic forces, and so on, which may in turn require modification of the structural system (e.g. increased member capacities). In many of the simplified design procedures currently used in practice, many of the steps indicated are implicitly incorporated into the design assumptions or performed by the designer on the basis of intuition, judgment, and experience.

The basic seismic-resistant design process considered consists of 4 phases: definition of structural environment; functional planning; prediction of structural behavior; and reliability analysis.

Definition of Structural Environment

In the first phase, the structural environment is defined. Of particular concern to seismic-resistant design is the nature of the design earthquake and the gravity loads, although other factors such as wind loading, ground settlement, fire potential, and so on must also be considered. Computers have been widely used in the establishment of design earthquakes, but this application will not be discussed in detail in this paper. Basic problems that must be considered in establishing such design criteria are discussed in Refs. 2 - 4. Computers have been used to assess the effects of local geology on the ground motions that might be experienced at a particular site [5]. Methods have also been developed to derive a ground motion that is critical for the elastic response of a particular structure [6].

Functional Planning

In the second phase, basic functional requirements are identified, design criteria to ensure the satisfaction of these requirements are established, and a basic conceptual design that will meet these general objectives is formulated. Computer-generated parametric studies can be valuable as guidelines at this stage to help the designer assess the overall response characteristics of various types of structural systems. A limited number of such studies have been completed for single-story [7-9] and multistory [9-12] buildings for a variety of modeling assumptions. Additional studies of this type are desirable.

Prediction of Structural Behavior

In this phase, a structural design is obtained iteratively by comparing the analytically-predicted behavior of a succession of trial designs with the basic design criteria established in the second phase. The third phase consists of a number of steps (Fig. 2). The first of these is to idealize the actual structure into a realistic mathematical model capable of being

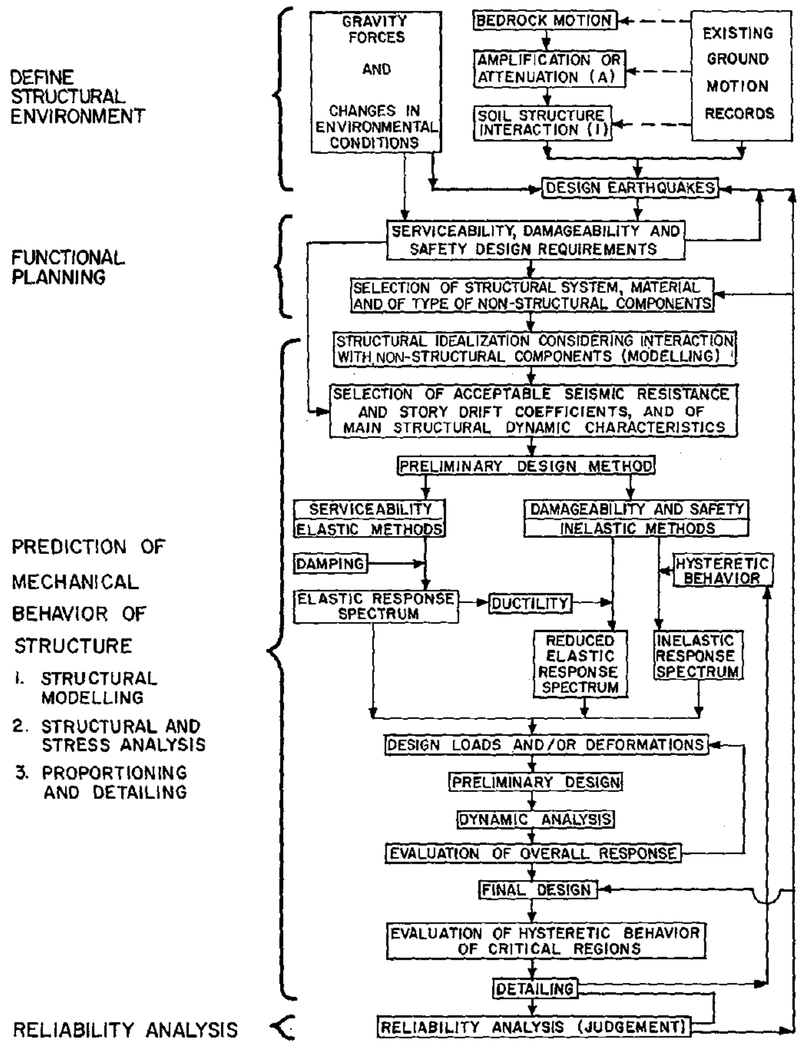

FIG. 2 FLOW DIAGRAM OF A GENERAL SEISMIC-RESISTANT DESIGN PROCESS

analyzed. Once this is done the basic design criteria must be expressed in terms of parameters that can be related to the structural idealization. The implications of this step will be discussed in more detail subsequently.

Preliminary design (proportioning) of members can be performed in a number of ways. In discussing design methods it is useful to introduce the concept of limit states [13]. A limit state refers to the state of a structure when it ceases to fulfill its functions or to satisfy the criteria for which it was designed. Such critical states are generally grouped into 2 broad categroies: serviceability and ultimate (or safety) limit states. Limit state design formats encourage the use of probabilistic methods, and Sawyer [14] has suggested a comprehensive design procedure in which the resistance of a structure at its various limit states is related to the probability that excitations capable of producing those failure states would occur. In this manner, the total life cycle cost of a structure may be minimized. Since the parameters and structural characteristics controlling each of the limit states will generally be different, it is useful to discuss each pertinent limit state separately.

The seismic-resistant design philosophy promulgated in the Recommended Lateral Force Requirements and Commentary of the Structural Engineers Association of California [15] considers 3 limit states, i.e.:

> "... structures designed in conformance with the provisions and principles set forth herein should be able to:
>
> (1) Resist minor earthquakes without damage
>
> (2) Resist moderate earthquakes without structural damage but with some nonstructural damage
>
> (3) Resist major earthquakes of the intensity of severity of the strongest experienced in California, without collapse, but with some structural as well as nonstructural damage.

For convenience these 3 limit states will be referred to subsequently as serviceability, damageability, and safety states, respectively. Current UBC [16] and SEAONC [15] design recommendations attempt to satisfy the requirements of all 3 limit states by imposing a single set of equivalent static lateral forces on an elastic structural idealization in conjunction with a number of design and detailing requirements. Such single-state design procedures are convenient for manual computation and have generally achieved satisfactory seismic safety. Current UBC [16] provisions require explicit consideration of the dynamic characteristics of complex, irregular, or unusual structures. In such cases, or when it is desirable to derive (using parametric studies) design forces for use in simplified preliminary design procedures for standard types of structures, the 3 limit states should be considered explicitly. Design forces (or deformations), design methods, and structural idealizations consistent with each limit state should be employed. These procedures will be discussed in a subsequent section.

Once a preliminary design is formulated it should be evaluated to determine whether it satisfies the design criteria established for each limit state. If it does not, the design must be modified and the process repeated. Analytical methods appropriate to evaluate the structural behavior for each limit state will be subsequently discussed.

When the basic proportions of the structure are determined, the critical regions must be detailed to withstand the forces and inelastic deformations that they are likely to suffer during severe ground motions, and the nonstructural elements must be detailed to minimize damage that they might suffer during moderate seismic excitations. To do this effectively, the designer must have meaningful indicies of the damage-producing deformations in nonstructural elements and of the forces and inelastic deformations that may develop at various critical regions in the structural components. It is also necessary to have reliable methods for determining the ability of nonstructural and structural details to sustain the deformations and forces that they may develop. Analytical methods for predicting the behavior of critical regions will be discussed later.

The reliability of the final design should then be assessed and compared with other feasible designs. In most cases this is currently done on the basis of engineering judgement and initial construction costs. Methods accounting for earthquake hazard probabilities, structural behavior predictions, and economic as well as other losses associated with probable damage have also been formulated [17]. However, very few practical applications of these methods have been attempted (e.g. Ref. 18), partly because of the expense involved, the large number of variables that should be considered for actual structures, the difficulty in associating costs with damage, and many other technical and political problems [17]. However, such methods provide a rational basis for assessing seismic risk and for increasing the efficiency of structural designs. These methods will undoubtedly become more common as computers find greater utilization in design.

PRELIMINARY DESIGN METHODS

As discussed previously, at least 3 limit states should be considered in a comprehensive design process. These are serviceability, damageability, and safety. Suggestions have been advanced for designing a structure for 2 limit states using a dual-spectrum criterion for seismic loading [19]. Member 'damage threshold' and structural 'collapse threshold' seismic events were defined. A number of existing buildings were redesigned using these criteria, and they were analyzed elastically to assess the impact of this type of design philosophy on structural costs and performance [19].

Since different types of structural behavior is expected in each state, different methods of design would be appropriate in each. Methods for achieving a balanced design that optionally satisfies the requirements of all 3 limit states simultaneously have not been adequately researched.

Serviceability

Under minor earthquake ground motions that might occur frequently during a structure's service life, no damage should be accepted. In this case, the structure would behave essentially in the elastic range. Thus, the design earthquake might reasonably be defined in terms of an elastic response spectrum and design forces derived using well-known procedures [4]. Standard design practices could then be used to select member sizes. Considerable work has been conducted by individual organizations to automate this elastic design process using computers of various sizes and capabilities [20]. While numerous applications of various optimization procedures have been applied to steel structures to minimize weight and control drifts under equivalent-static lateral seismic forces [21-23], few corresponding examples can be found for reinforced concrete structures.

Damageability

Considerable economic losses during moderate earthquakes can be attributed to damage to nonstructural elements [24,25]. Definitions of this limit state was formalized in Ref. 26. Quantitative design indices to account for damageability as well as the other limit states have also been introduced [26]. However, data related to damage-producing thresholds for various nonstructural elements is not extensive at present, and consequently design for this limit state remains largely judgmental.

Safety

Under severe ground shaking the structure must not collapse. Economic considerations generally require that some of the energy input during such events be dissipated by large, yet controlled inelastic deformations. In such cases the structure might be reasonably expected to suffer significant structural and nonstructural damage. Most current design methods consider safety by using elastic structural idealizations and design forces considerably smaller than those that would likely develop in the structure during a major earthquake if it remained elastic. A number of methods for specifying reduced design forces have been suggested to allow the designer to control the expected amount of inelastic action. Most of these are based on the computed dynamic response of ideal single degree-of-freedom (SDOF) systems assuming elasto-perfectly plastic structural behavior (e.g., see Ref. 28). Recent analytical studies [2] indicate that the ground motion characteristics controlling the response of yielding structures are different from those that govern the behavior of elastic structures. On the basis of these results, special care should be exercised in establishing design earthquakes for safety limit states if a structure can be exposed to excitations producing large inelastic deformations (e.g. structures sited near potential fault ruptures).

Another fundamental problem is to establish methods for deriving the distribution and magnitude of the equivalent static lateral design forces for inelastic multiple degree-of-freedom systems. Inelastic design response spectrum are generally derived for SDOF systems. Since the principle of mode superposition is not applicable for structures that respond in the nonlinear range, it is not clear whether reliable design forces can be obtained using inelastic design response spectra and elastic modal characteristics in a manner analogous to that used for elastic spectral design [4]. Research is needed to determine the applicability of such methods and to develop improved methods for deriving inelastic design forces for multiple degree-of-freedom systems.

SEISMIC ANALYSIS PROCEDURES

When there is sufficient experience with the seismic performance of particular types of structural systems during past earthquakes, or where the seismic exposure or hazard is low, it may not be necessary to perform detailed seismic response analyses. In such cases, it is desirable to construct a structural system for reasonably high seismic forces and to provide it with a large energy dissipation capacity. Current codes [16] require evaluation of the dynamic response characteristics only for complex or unusual buildings. Whenever possible, however, the adequacy of the preliminary structural design should be analyzed to determine if it satisfies the design criteria for each limit state. The analysis methods used in each case should be consistent with the anticipated structural behavior in each of the limit states. For example, linear elastic structural idealizations may be appropriate for serviceability checks, but nonlinear response idealizations may be required for damageability or safety limit state analyses.

Serviceability

Many general computer programs have been developed to analyze elastic structural systems [33,34]. A number of programs have been written to take advantage of common structural features encountered in many building systems and to have computer-designer interfaces appropriate to the design of buildings [35,36].

For example, the computer program *ETABS* [36] idealizes buildings as a system of independent, vertical frames interconnected at each floor level by diaphragms that are rigid in their own plane. Frames need not be planar, and they can be located arbitrarily in plan. Horizontal beam element, vertical column elements, diagonal bracing elements, and shear (infill) panel elements may be used to model the frames. In general, the elements allow consideration of a wide variety of factors encountered in building structures: distributed loads on beams; axial shearing and bending deformations in columns; rigid beam-column joints; etc. Various combinations of vertical and lateral static loading may be considered. Response may be determined for a single horizontal component of ground motion specified in terms of an acceleration time-history or as an acceleration response spectrum. A combination of convenient input/output formats, generally reasonable structural idealizations, and program efficiency has led to significant professional utilization of such programs.

The addition of a number of features would improve the usefulness of such special purpose programs. For example, options for automatic comparison of computed member internal forces with code permitted values and for expanded output capabilities (including computer graphics) would assist in interpreting the analytical results. Incorporation of procedures that permit automated redesign of the structure or detailing of members would also be valuable.

The applicability of such programs could also be broadened by incorporating certain additional features. Some of these are:

(1) Consideration of the vertical as well as the 2 horizontal translational components of ground motion;

(2) Capabilities to account for in-plane floor diaphragm deformations;

(3) Capabilities to consider multiple floor diaphragms at a level to permit considerations of structures having several penthouses, that branch into more than one tower above a certain level, etc; and

(4) Capabilities to consider realistically pierced structural walls or structural cores (possibly using finite element and substructuring techniques [36]).

No matter how refined the analytical method, the reliability of the results obtained depends on the accuracy with which the structural and nonstructural elements are modeled. Problems with modeling reinforced concrete buildings are particularly complex. (For a discussion on some of these see Ref. 29.) Improved methods for modeling floor systems [37,39], structural cores [37,40], and nonstructural elements [41] have recently been developed, but further refinements are needed. Guidelines for selection of damping values should be expanded based on the type of structural system, nonstructural elements, and the severity of excitation.

Damageability

Analytical methods to account for the damage to nonstructural components have not been extensively developed. Where nonstructural elements are not expected to contribute significantly to the structural response, they are generally disregarded. In this case, the damage to nonstructural elements is estimated in terms of structural response parameters, generally floor accelerations and story drifts. There is very little reliable data available to make such interpretations more than qualitative estimates of damage. Furthermore, horizontal story drifts as typically computed may not adequately represent the true damage potential in multistory structures [38]. In some structures, a considerable portion of the horizontal displacement may result from column axial deformations, as shown in Fig. 3. A better index of nonstructural damage to panel shearing type of deformations might be the tangential story drift index, R, (Fig. 3) which can easily be incorporated in most computer programs.

Where nonstructural elements may effect structural response, they should be included in the structural model. However, few realistic models exist to idealize the nonlinear behavior of common nonstructural elements. Computer-based studies of the nonlinear dynamic response of an elastic infilled frame have been reported [41] for the case where the frame and infills were initially separated by a small amount. Idealizations based on diagonal braces to represent the nonlinear mechanical characteristics of structural masonry infills have been recently developed [42].

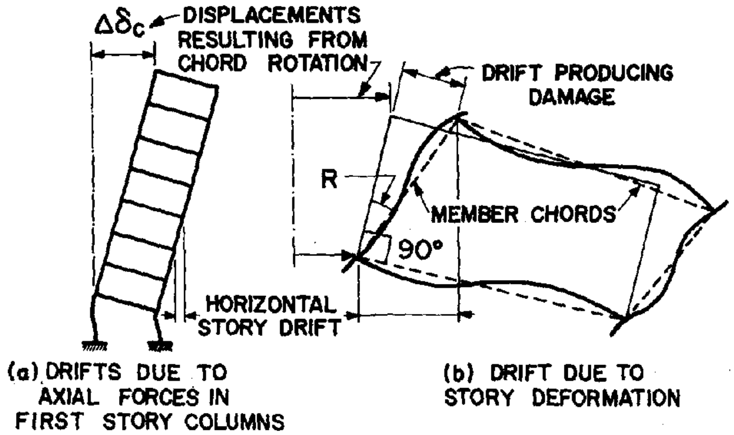

FIGURE 3. DEFINITION OF TANGENTIAL STORY DRIFT INDEX, R

Safety

Since structures are generally designed to sustain significant structural as well as nonstructural damages when exposed to a major earthquake, it is usually necessary to use nonlinear analysis techniques to predict their behavior. It has been shown that interpretation of results of elastic analyses to estimate inelastic behavior is difficult and, in many cases, unreliable [29]. The following discussion will focus on 3 main topics: mechanical models for idealizing nonlinear structural behavior; available computer programs in which such models are implemented; and evaluation of analytical results.

Nonlinear mechanical models. - Two different approaches have been attempted to account for the stiffness degradation that may occur in multistory buildings during earthquakes: a shear building idealization and a discrete member idealization.

The first approach to this problem has been to represent the gross interstory shear deflection characteristics of a structure by an analytically or empirically derived model which incorporates some type of stiffness degradation. In this case, the actual structure is idealized as an inelastic shear building. Several investigators have conducted parameter studies of single-story [7-9] and multistory [9,10] shear buildings, and the method has been widely used for design [9,43] and seismic damage studies [44-46].

This shear building idealization is very attractive for high-rise buildings because the computational efforts required to consider the effects of inelastic behavior and multi-dimensional structural response is relatively small. The principal disadvantages of the method, however, are the difficulty encountered in realistically modeling tall buildings as shear structures and the problems involved in determining the required interstory shear-drift relationships.

The second approach has been developed to account for the behavior of individual structural members. The approach is based on simplified nonlinear member mechanical characteristics which make it possible to estimate the magnitude and distribution of inelastic deformations throughout a moderately-sized structure with reasonable computational effort.

A common idealization used for flexural members has been that of concentrating the inelastic deformations, when they occur, at the ends of the member; in doing this, Clough et al. [47] assumed a bilinear hysteretic moment-curvature relationship which resulted in a two-component, parallel element model that has found considerable application in the analyses of both steel and reinforced concrete structures. Walpole and Shephard [48] and Mahin and Bertero [29] have developed computer programs to evaluate the seismic performance of planar reinforced concrete frames using elements based on bilinear hysteretic moment-curvature relationships. This idealization disregards the stiffness degradation that may occur under moment reversal.

Several investigators have formulated more refined mathematical models to account for the stiffness degradation that may occur in flexural members. Generally, inelastic deformations have been assumed to occur only in regions located at the ends of an element. In such cases, the flexural stiffness properties may degrade over 2 separate regions of the member due to prior inelastic actions. Consequently, calculation of the element stiffness is more complex than for the shear building analogy since indices of inelastic deformations at 2 separate locations must now be considered in the mathematical model.

On the basis of beam-column subassemblage tests, Imbeault and Nielson [49] have formulated a degrading stiffness model in which the primary stiffness of a conventional bilinear hysteretic model, K, is reduced as a function of the maximum system displacement, D_{max}. The primary stiffness of the resulting degrading bilinear hysteretic model is given by:

$$K = K_o \left[\frac{D_{yield}}{|D_{max}|}\right]^{\alpha} \qquad \text{for } |D_{max}| \geq |D_{yield}| \tag{1}$$

where K_o is the initial elastic stiffness, D_{yield} is the yield displacement, and α is an empirical constant. The strain-hardening stiffness has a constant value. This model was applied to bilinear hysteretic flexural elements by using a norm of the inelastic deformations at each end of the element as an index of displacement, and by varying the primary slope of the element's governing moment-curvature relationship according to Eq.(1). Anderson [50] has shown that this model can be incorporated into computer programs based on conventional two-component flexural elements with negligible increase in computational effort. It is not clear, however, whether such idealizations remain applicable if the inelastic deformation expected at each end of a member are greatly dissimilar, or if substantial degradation of stiffness occurs due to large inelastic deformations.

Yoshioka, Takeda and Nakagawa [51] have formulated an analytical model in which an elastic flexural element is connected to the rest of the structure by nonlinear rotational springs at each end. The moment-rotation relationships which control these springs include both yielding and stiffness degradation according to the Takeda degrading stiffness model [52]. Although this approach explicitly accounts for the degradation of stiffness that may occur at each end of the member, the appropriate spring stiffness may be difficult to establish in practice. Powell [53], and Guendelman-Israel and Powell [54] have implimented this type of element in general-purpose nonlinear dynamic analysis programs.

Otani [55] has suggested that a flexural element may be analytically separated into 2 cantilever elements at the point of inflection. By using 2 independent semi-empirical degradation laws to determine the stiffness of each of the 2 resulting cantilever elements, it was possible to reconstruct the element's flexural stiffness. Although good agreement with experimental data has been reported, a number of undesirable features have been observed (e.g. unsymmetric stiffness coefficients, solution instability, etc.).

Umemura and others [56] have developed a comprehensive computer program that accounts for the stiffness degradation occurring in beam-columns, joints, and shear walls. The method is based on an assumed parabolic distribution of flexural and shear stiffness along flexural members. The effective sectional stiffness (EI) at the ends of the member is based on a trilinear hysteretic stiffness degrading model, and the initial sectional stiffness is assumed at the point of inflection. Substantially improved results, compared to comparable shear building analyses, have been reported by these authors.

Mark [54] has developed a computer program to compute the static and dynamic response of slender flexural elements based on a 'fiber' model that accounts explicitly for the nonlinear stress-strain behavior of the various layers of steel and concrete across the depth of a section. While results reported for this model are in better agreement with experimental results than some other simpler models studied, the author reports a number of theoretical and practical deficiencies with the model. Shearing deformations, bond deterioration, and joint rotation due to slippage of anchorage reinforcement have not been accounted for. The method must consider very small deformation increments in order to achieve accuracy so that it may not be an economical or practical design tool. However, methods such as these when experimentally verified would be valuable in evaluating special details or structures and in developing and assessing simpler analytical models that could be practicable for design.

Difficulties are also encountered in modeling the nonlinear characteristics of structural components such as columns, structural (shear) walls, and floor systems. The cyclic hysteretic behavior of reinforced concrete members subject to combined bending and axial load [57] is generally much more complex than can be represented by existing simple models. The behavior of structural walls is also complex [58], but it is not certain whether simple one-dimension models or even complex laminar models would be adequate due to the large contribution of diagonal cracking and shearing deformations to the overall deflections. The assumptions used in modeling floor systems can have a large effect on the computed structural response [29]. While there are methods [39] for estimating the elastic stiffness characteristics of some types of floor-slab systems, modeling of their nonlinear mechanical characteristics is difficult [59]. Further development and experimental corroboration is needed for these type of structural components.

Available computer programs. - In addition to the programs developed to impliment the models discussed above, several general purpose nonlinear programs have been written. Representative of those based on bilinear hysteretic moment-curvature relationships is *SERF* [29] which incorporates a number of features appropriate for modeling R/C structures such as non-symmetrically reinforced sections and realistic column axial force-bending moment interaction curves. The structure is idealized in this program as a planar assemblage of horizontal, axially inextensible beam elements, vertical column elements, and diagonal bracing elements. Finite-dimensioned beam-column joints are assumed to be rigid. Gravity loads and horizontal and vertical ground accelerations can be considered. The simplicity of such idealizations permits computational efficiency, but problems such as unusual structural geometries, stiffness degradation, and torsion cannot be accounted for.

General purpose computer programs such as *DRAIN-2D* [53] solve some of these problems by not limiting the structural geometry and by incorporating a larger number of nonlinear element models. For example, this program includes a stiffness degrading beam-column element as discussed previously, in addition to bilinear hysteretic beam-column elements, diagonal bracing elements, shear panel elements, and flexible joint elements. New element models can be easily added. However, this program is still limited to two-dimensional structural idealizations, and its input/output options are oriented to general, rather than building, types of structures.

An increasing number of general-purpose three-dimensional nonlinear dynamic computer programs have been developed recently. For example, *DRAIN-TABS* [54] idealizes a structure as an array of vertical planar frames arbitrarily oriented in space. Individual frames are modeled in a similar fashion to the program *DRAIN-2D*. Horizontal displacements of joints in different frames can be kinematically related to achieve a rigid in-plane floor diaphragm effect, but the vertical and rotational displacements of joints common to 2 frames cannot be coupled in the program. Thus structures where such coupling is important (e.g. tubular frame buildings) cannot be modeled adequately with this idealization. An approximate procedure has been incorporated to account for the effect of axial load on the moment capacity of columns common to more than one frame. Vertical and 2 horizontal components of ground motion may be considered in addition to static gravity loads. This program permits the response analysis of many types of complex structural systems, but it cannot account for coupling of nonplanar frames, and it uses approximate techniques to account for so-called P-Δ effects.

More analytically versatile three-dimensional programs have been developed, such as *ANSR* [60], but these generally do not now have extensive building-related element libraries. Expansion of these element libraries to include basic material models (e.g. multiaxial concrete models, steel models, and bond-slip models) in addition to simpler nonlinear member idealizations would result in powerful research tools that could be used to formulate simplified numerical techniques consistent with design requirements and to assess the reliability of simplified mechanical idealizations.

From the perspective of design application, it would be desirable if the computer programs used for safety limit-state evaluations could also efficiently check serviceability conditions. Alternatively, the input data should be similar for the programs used for these different state evaluations. This would reduce the costs and the possibility of errors in preparing 2 sets of data. Even where serviceability checks are not necessary, elastic analyses are often required to determine mode shapes and periods. Another useful, though not generally included, program feature would be a restart capability. This would permit inexpensive data checks, execution time estimates, some capabilities for manual modification of structural parameters during execution, and consideration of post-earthquake events such as aftershocks, fire, and repair.

Where large or complex structural systems are to be analysed, the cost of the analysis may be prohibitive using conventional solution techniques. Investigations into methods for increasing the efficiency of such analyses (e.g. use of substructuring techniques, development of 'macroelements' [21], iterative methods, etc.) should be conducted.

Evaluation of results. - A problem inherent with all nonlinear analysis methods is the difficulty in interpreting results. This problem has 3 aspects: (1) identification of critical response parameters that should be output; (2) presentation of these response parameters; and (3) evaluation of response parameters in terms of actual structural behavior.

As discussed in Ref. 38, different response parameters are needed to evaluate various aspects of building response. For example, in assessing overall response, it is useful to know the maximum floor level displacements, accelerations, and overturning moments, the story (or local panel) drifts, and the story shears and torques. Provisions for extracting such information should be included in building-oriented computer programs. Parameters presented regarding individual member behavior should be selected on the basis of their ability to reflect the actual behavior of the member, especially when simplified member idealizations are used. In addition to maximum internal forces, realistic indices of the peak and cumulative inelastic deformations and the number and severity of inelastic reversals are needed. A discussion of problems in defining such indices is contained in Refs. 29 and 38.

The large quantity and complexity of the results obtained with nonlinear computer programs necessitates an output format that facilitates interpretation of the results and evaluation of the structural performance. Considerable development of output capabilities (including computer graphics) is needed.

The results computed on the basis of simple analytical models must be interpreted in terms of the expected behavior of real structural elements subjected to similar loading and/or deformation histories. To do this it is necessary to have empirical or analytical data related to the inelastic deformation capacity of members. While there is considerable experimental information related to the hysteretic behavior of various types of members, these results are not generally expressed in terms of nondimensionalized or other parameters that can be applied to other situations. For example, experimentally obtained ductility factors based on displacement cannot be used to assess analytical results obtained for structures with different geometries. Also, little information is available relating the cummulative energy dissipation capacity to the loading history. Thus, it is important to coordinate analytical and experimental research objectives, and to express response parameters in consistant, physically meaningful ways in both experimental and analytical investigations.

DETAILING

Members must be detailed to withstand the internal forces and inelastic deformations that may be required of them in their various limit states. Computers can be advantageously used at this stage in the design process. First, computers can be used to impliment standard design details such as those required by building codes [61]. Secondly, computer programs can be developed to simulate the behavior of specific design details. The capability to predict reliably the local behavior of a critical region accounting for its detailing for various loading conditions permits: (1) reconciliation of force and/or deformation demands for various critical regions predicted in structural response analyses with an analytical-derived capacity estimate based on the actual detailing; and (2) evaluation and improvement of standard details without the expense of experimental investigations. Current procedures for numerically simulating the behavior of critical regions subjected to uniaxial bending and bending in combination with axial load and shear will be briefly examined in the following.

Constitutive Relationships

The accurate representation of the stress-strain relationships for concrete and reinforcing steel and of the bond stress-slip relationship governing the interaction of the 2 materials are crucial to predicting member behavior. Steel is commonly idealized as being elasto-perfectly plastic. Several investigators [57, 62-65] have formulated more realistic analytical models for the hysteretic stress-strain behavior of reinforcing steel. These differ substantially from the elasto-plastic idealization, especially under large strain reversals. It has been shown that the flexural characteristics of reinforced concrete sections are very sensitive to the details of the hysteretic behavior assumed for the reinforcement under monotonic loading [29,62] and especially under strain reversal [57,63]. Therefore, these analytical models must be refined as much as possible to reflect actual reinforcement properties.

Confined concrete in reinforced concrete flexural members is under complex multiaxial states of stress [66]. A number of uniaxial stress-strain models have been formulated accounting for the amount of transverse reinforcement under monotonic loading [66-68] and strain reversal [57,63,66]. It appears that the quantity of confinement provided has a substantial effect on the concrete stress-strain relationships, and further research is necessary to define these relationships. However, studies [57,63] indicate that the details of the assumed relationships for loading and unloading do not generally have a significant effect on computed moment-average curvature relationships.

The typical kinematic assumption of linear strain distribution is not strictly valid in beams because of the bond deterioration that occurs in the vicinity of cracks. Substantial experimental and analytical research is currently being conducted in this area [63], but few reliable analytical models have been developed thus far.

Moment-Average Curvature Relationships

A number of computer programs have been developed to predict the moment-average curvature relationships that would develop in flexural members assuming linear strain distributions for monotonic loading [29,69] and load reversal [57,62,63,66]. Some of these can account for spalling of unconfined concrete [29]. Results of analyses conducted with such methods indicate that significant errors can result in the predicted behavior if realistic material properties are not used. For example, use of specified minimum material strengths and simplified stress-strain relationships can significantly underestimate the moment capacity of a section and therefore the actual shears that might develop in a member [29]. Studies also indicate that confinement of the concrete by closely spaced ties can substantially increase the maximum curvatures that a section can develop. However, there is very little experimentally derived data to indicate when buckling of the longitudinal reinforcement would occur or what value of maximum concrete strain (or section curvature) can be developed as a function of the concrete properties, the amount and location of longitudinal and transverse reinforcement, shear span ratio, and so on. Consequently many programs assume failure occurs when the flexural strength drops below an arbitrary fraction of the ultimate capacity [62] or when the solution algorithm fails to converge [69]. To estimate the maximum curvature that a section can reliably develop, research must be directed to experimentally refine failure criteria for such members.

Effect of Axial Load

While many of the programs for monotonic loading conditions discussed in the previous section have the capability of including nonzero axial loads, fewer programs are available to compute the effect of curvature reversal on the hysteretic behavior of such members. Under monotonic loading, axial compressive forces reduce the maximum curvature the section can develop and generally produce a substantial drop in moment capacity once spalling initiates [29]. Studies of columns indicate much more complex behavior [57,70] on deformation reversal than is usually exhibited for zero axial load, especially for cases where moment and axial force vary simultaneously [57]. Further analytical refinements are necessary to investigate the required amount of transverse reinforcement necessary to confine the concrete core and laterally restrain longitudinal reinforcement so that premature failure will not occur, and to develop better member mechanical idealizations for overall structural analysis programs.

Effect of Shear

It is desirable that members be detailed to fail in a ductile flexural mode rather than brittly in shear. Consequently it is necessary to proportion the transverse reinforcement in flexural members to resist the maximum shear forces that would actually be developed. A recent computer study [59] of code-based shear reinforcement requirements [16] indicates that there are a number of ambiguities in applying code specifications to realistic sections and points out the need for carefully modeling material properties and member boundary conditions when studying shear.

A substantial portion of the deflections in reinforced concrete members can be contributed by shearing deformations. In an experimental and analytical study of this problem, Ma, Bertero, and Popov [63] developed a simplified computer-implimented model to predict the shear force - shear deformation hysteretic relationship of reinforced concrete beams. This model included the effects of aggregate interlocking, stirrup-tie resistance, dowel action, and the shear resistance of uncracked concrete. Reasonable correlation of experimental and analytical results were reported, particularly for the initial loading stages. Further refinements in the modeling of aggregate interlocking, dowel action, and bond deterioration were suggested to improve such models.

CONCLUDING REMARKS AND RECOMMENDATIONS

A general overview of the use of computers in the design of reinforced concrete buildings has been presented. Computers are being applied in a wide variety of ways to the design process, and there has been a very rapid increase in computational capabilities during the past few years. In order to coordinate future research efforts and to avoid duplication, it would be desirable to conduct a comprehensive survey [71] of available computer programs and their capabilities.

A number of theoretical perspectives for evaluating the overall design process (seismic risk analysis, limit states design, and comprehensive design) have been reviewed. The efficient implimentation of these concepts depends heavily on the ability to predict structural behavior under various loading conditions. The development of reliable and economical computer-based simulation procedures will not only give designers greater confidence in their design decisions but may allow for the practical implimentation of these more quantitative design evaluation and decision procedures.

Full realization of the potential for computers in the design process depends partly on the education of the design profession to the design capabilities of computers but also on the development of versatile, economic, and reliable computer programs that are design oriented. Most computer software development has been oriented towards the basic analytical problem of predicting seismic response. There is considerable need to present the results of such analyses in a form that is easily interpretable in terms of expected building performance and to develop computer capabilities to redesign structures or detail members on the basis of analytically obtained results.

Computer-automated design procedures which impliment standard design recommendations are becoming more common. Considerable research is needed on how to formulate preliminary design proportions based on the expected design environment and the various limit state requirements. Development of optimation procedures for elastic and inelastic systems subjected to generalized earthquake-like loading is desirable. Methods to specify reliable design forces for inelastic multiple degree-of-freedom systems should be investigated.

Dynamic response capabilities are generally quite extensive for elastic systems. Comparable capabilities are desirable for structures that respond in the inelastic range. Experimental and analytical research is needed to formulate and verify mathematical models for the behavior of most reinforced concrete structural components. The reliable modeling of structural systems remains one of the most significant problems in analysis. Methods for accounting for nonstructural elements are needed. Development of I/O capabilities to assist users in preparing data and interpreting results are needed.

The detailing of critical regions portends substantial opportunities for applying computers to design. Considerable research is being conducted to formulate mathematical models for predicting local member behavior. Establishment of realistic constitutive relationships, particularly for the reinforcement and for the bond between concrete and steel is particularly important. Experimentally derived failure criteria for members are needed based on the detailing of the region and loading history. Methods to include the effect of shear and axial force are needed for flexural elements. Studies of the nonlinear mechanical characteristics of structural walls and floor slab systems are also desirable.

Analytical investigations related to a particular phase of the design process should be carefully integrated and coordinated with the requirements of, and studies on, other design phases in order to optimize their utility. For example, it would be desirable that analytical investigations related to the overall response of a structure and those related to prediction of the inelastic deformation capacity of individual critical regions use similar response parameters so that results can be compared. It is also necessary to integrate experimental and analytical investigations. Not only is it desirable to corroborate mathematical models with experimental data and to devise realistic experimental loading (deformation) histories using realistic mathematical models, but analytical and experimental response parameters should be expressed in a consistent and nondimensionalized manner in order to facilitate interpretation of results.

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