The EERC-CUREe Symposium in Honor of Vitelmo V. Bertero

January 31 - February 1, 1997

Berkeley, California

Award from Instituto Universitario di Architettura de Venezia, 1965
Elected to the Academy of Science of Argentina, 1971
Tai Krishna Award from India Society of Earthquake Technology, 1974
James F. Lincoln Arc Welding Award, 1975
Life Member of the Indian Society of Earthquake Technology, 1975
Fellow of the American Concrete Institute, 1975
Shared the ASCE-ACI Raymond C. Reese Award, 197[illegible]
Trans[illegible] ASCE, 197[illegible]
Distinguished Guest of the City of Mérida, Venezuela and
Diploma [illegible] Mérida in recognition of [illegible] Earthquake [illegible]
Honorary Member, [illegible] Club of [illegible]res, Novemb[illegible]
Honorary Professor of the University of Guayaquil, Ecuador, 1979
Honorary Member of the Structural Engineering Association of the Guayas, Ecuador, 1979
Commendation from the Mexican Society of Seismic Engineering, 1983
Honorary Professor of the University of Rosario, Argentina, 1983
Honorary Advisor, Research and Development Institute, University of Rosario, Argentina, June 1983
Diploma of Recognition, "Contribution of Earthquake Engineering",
Mexican Society of Seismic Engineering, Mexico, November 1983
ACI Raymond C. Reese Structural Research Award (Diploma and Medal), 1986
Extraordinary Chair "Ingeniero Javier Barrios Sierra," College of Engineering,
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First Honorary Member, Sociedad Venezolana de Ingeniería Sismica, 1987
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ASCE Moisseiff Award, [illegible]
James F. Lincoln Arc Welding Foundation Best Program Award, 1987
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Roy W. Carlson Distinguished Professor of Civil Engineering, University of California, Berkeley, 1989-[illegible]
Shared the ASCE J. James R. Croes Medal, 1989
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Elected Member of the Argentinean Academy of Engineering, "Academia Nacional de Ingeniería," 1989
Awarded the Torroja International Diploma and the
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T. R. Higgins Lectureship Award, American Institute of Steel Construction, 1990
Byron L. and Elvira E. Nishkian Chair of Structural Engineering,
University of California, Berkeley, July 1990 - June 1996
Appointed Honorary Professor, University of Buenos Aires, 1990
Shared the ASCE Nathan M. Newmark Medal, 1991
Awarded the University of California, Berkeley's highest honor, the Berkeley Citation, 1991
Appointed First Shimizu Visiting Professor in the Department of Civil Engineering, Stanford University, 1991
Construction Innovation Forum's NOVA Award, 1991
Appointed Honorary Professor, Facultad Regional Mendoza, Argentina, 1992
Honorary Member, Structural Engineers' Association of Argentina, 1992
Doctorado Honoris Causa en Ingeniería from the Universidad de Los Andes, Mérida, Venezuela, 1992
Medal and Order "Dr. Julio Febres Cordero," Estado de Mérida, Venezuela, 1993
Awarded "Diploma al Mérito Konex 1993" in Science and Technology, Konex Foundation of Argentina
Housner Medal, Earthquake Engineering Research Institute, 1995
Named Honorary Member of Structural Engineers Association of Northern California, 1996

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A Symposium in Honor of Vitelmo Bertero, highlighting his distinguished career and engineering achievements, was held January 30-February 1, 1997, in Berkeley, California. Professor Bertero is professor emeritus of civil engineering at the University of California, Berkeley, and a former director of EERC. More than 250 fellow engineers and colleagues and former students attended the symposium. Thirty-seven symposium papers are included in this report. These papers focus on many of Professor Bertero's areas of interest: reinforced concrete structures, structural steel, design methodologies, dynamic structural analysis, lessons learned from earthquakes, and improving seismic building codes.

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The Earthquake Engineering Research Center wishes to acknowledge and thank the individuals and organizations that contributed to the EERC-CUREe Symposium held to honor the achievements of Professor Vitelmo Bertero of the University of California at Berkeley.

The Symposium was organized by a Steering Committee comprising Dr. James Anderson (University of Southern California), Dr. Helmut Krawinkler (Stanford University), Dr. Stephen A. Mahin (University of California at Berkeley), and Dr. Andrew Whittaker (University of California at Berkeley). Drs. Anderson, Krawinkler, and Mahin deserve much of the credit for the technical program.

Six organizations provided financial and in-kind support to the Symposium, namely, the California Universities for Research in Earthquake Engineering (CUREe); the National Science Foundation (Dr. S.C. Liu, Program Manager); Degenkolb Engineers, San Francisco, CA (Mr. Chris Poland, Senior Principal); Fluor Daniel, Inc., Irvine, CA (Mr. Bob Bachman, Engineering Manager); the Japan Structural Consultants Association (Mr. Yoshio Murata); Mr. Nagahide Kani, and the Earthquake Engineering Research Center at the University of California at Berkeley. This support was used to support young faculty from within the United States to attend the Symposium, meet travel costs for some speakers, and publish these proceedings. This support is gratefully acknowledged.

Many of the speakers and moderators traveled at their own expense to the Symposium to honor their friend, and often mentor, Vitelmo Bertero. Their time and effort is much appreciated.

Other individuals are deserving of special thanks. Bob Reitherman, Parshaw Vaziri, and Wanda Realegeno of CUREe, and Carol Cameron, Betty Marton, Barbara Mauk, and Darlene Wright are credited with organizing the Symposium and the preparation of this report. We sincerely thank them for their tireless efforts.

Andrew Whittaker and Jack Moehle
Earthquake Engineering Research Center
January 1997

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Vitelmo V. Bertero

Drift-Driven Design for Earthquake Resistance of Reinforced Concrete

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WEST LAFAYETTE, IN

SUMMARY

It is suggested that reinforced concrete structures should be proportioned on the basis of drift and then checked for lateral strength rather than being proportioned for strength and then checked for drift. A simple heuristic procedure for determining drift of reinforced concrete structures in the nonlinear range of response is described.

INTRODUCTION

The shift of the basis for earthquake-resistant design from strength to drift (lateral displacement) is an important, if not the most important, current issue. It is a fitting topic for discussion in the course of celebrating the achievements of Professor Bertero who has made very important contributions to earthquake-resistant design.

Design of a structure is the product of the designer's personal experience, creativity, and judgment. Calculations for design are impersonal and are controlled by the canons of the professional community. More often than not, the designer spends most of his/her time in shaping the calculations to fit the conceived design, sometimes defeating and sometimes being defeated by the calculation rules. To the credit of the structural engineering profession, designers generally have unfailing faith in the required calculations and trust that the labor and the bottlenecks of design calculations are always well worth the effort because they assure safety.

Almost all generally accepted design algorithms for earthquake resistance start with the definition, explicitly or implicitly, of a vibration period that leads to a base shear force coefficient suitable for the location of the building. Most of the labor of the exercise, loosely called "analysis," relates to strength. The procedure fits seamlessly into traditional calculations for gravity loads which are unquestionably related to strength.

Considering that structural failures in earthquakes have rarely been attributed to an insufficiency of base shear strength, it is curious that use of lateral force as the design base has not been questioned more often (Moehle, 1992). There are at least two explanations for the inertia. The first is that of habit. Safety for gravity loads is a matter of strength. So should it be for lateral forces, even if the lateral forces are arbitrarily determined and are to gravity forces, especially those related to self-weight, as apples are to oranges. A second explanation is that to determine drift requires more input data than usually available and involves more work than the anticipated accuracy of the result justifies. Even if all properties of the building are known, the characteristics of the ground motion are not.

Time and education will eventually erode the inertia associated with habit. This brief essay describes a simple procedure for determining drift and its development. The behavior of earthquake-resistant structures in reinforced concrete is controlled by the designer through adjustments of three ratios: strength to weight, stiffness to weight, and drift to height. The index values to those parameters are the base shear strength coefficient, period, and drift ratio. In the following sections, the interaction among these three indices will be discussed in a hypothetical environment, in relation to a single-degree-of-freedom (SDOF) oscillator of reinforced concrete with unbounded toughness. Simple as it is, the SDOF oscillator captures much of the behavior of reinforced concrete structures in relation to displacement response. The question of toughness is primarily a question of transverse reinforcement and detail. It is assumed that the building, for which the reinforced concrete oscillator is the metaphor, has the requisite detail to avoid brittle failure.

EXPERIMENTAL OBSERVATIONS

Testing reinforced concrete SDOF systems with the help of an earthquake-simulation system, Takeda (1970) observed an interesting trend in the measured drift maxima. Each of his test specimens was subjected to several base motions of increasing intensity. The same base motion was repeated in successive test runs amplitudes scaled by a factor that increased from run to run. The base motion caused yielding in the first run. In subsequent runs, the measured maximum lateral force tended to remain constant but the maximum drift increased more or less linearly with increase in base motion intensity. Takeda and his co-workers noted the trend but, being preoccupied with interpretation of the results in terms of a reduced-force model (reduced with respect to the force for linear response), did not emphasize that there was a direct relationship between ground motion intensity and maximum drift response and that the relationship appeared to be independent of strength.

The observation about the direct relationship between base-motion intensity and drift was repeated in a series of earthquake-simulation tests of multi-story frames by Otani (1973). Otani also observed that if a reinforced concrete structure, subjected to a base motion of sufficient intensity to develop yield, was tested for a second time with a base motion of similar intensity, it developed essentially the same maximum drift as it did in the first test. Maximum drift response appeared to be a function of the initial properties of the structure and not related to the stiffness of the structure at the beginning of the second test. This conclusion was decisively confirmed by other experiments (Cecen, 1979) but it had no visible effect on modeling of reinforced concrete structures for design or analysis. These tests also demonstrated that, barring story mechanisms, mode shapes in the nonlinear range of response were quite similar to those in the linear range of response.

INTERPRETATION OF EXPERIMENTAL OBSERVATIONS

Searching for a criterion for assessing performance of reinforced concrete buildings in earthquakes, Algan (1982) found the drift ratio to be the most pragmatic index. He suggested that, as long as brittle failure was avoided, ductility limits (toughness) of reinforced concrete structures were usually of secondary importance. By the time such limits were approached, the building would be a total loss because of the damage to its contents.

To proportion a structure in order to protect its contents, the most convenient vehicle was considered to be limitation of the drift ratio on the basis of information about fragility of the contents and tolerable loss. To incorporate the drift limit in the design process, it was necessary to develop a simple and direct way of estimating nonlinear drift because structures were typically designed for forces reduced from levels associated with linear response. Drift calculated on the basis of reduced forces would continue to send the message that drift was not a problem.

To calculate drift associated with nonlinear response required more information about the characteristics of the ground motion and of the building than would be available to the designer. To overcome that obstacle, Algan used the substitute-structure approach (Shibata, 1976) to estimate nonlinear drift response using linear-response spectra.

Figure 1 shows displacement spectra qualitatively for the nearly-constant acceleration and velocity ranges (Newmark, 1961 & 1970). Curves are shown for idealized response spectra at damping factors of 2 and 10%. The ordinates for the curve for 10% are set to be half of those for 2% (Shibata, 1976).

In terms of linear response spectra, Algan interpreted the changes in effective stiffness and equivalent damping as follows. Consider a structure with initial period T_i within the range of nearly-constant velocity response, and damping factor of 2%. It will have a spectral response displacement of D_1 as indicated in the figure. Subjected to a strong ground motion, the reinforced concrete structure will yield. Its period will increase to, say, 3T. Its effective damping will increase to, say, 10%. Thus, its maximum spectral displacement, D_2, is estimated to be $1.5\,{}_1D$. This is a convenient vehicle for estimating nonlinear displacement. The displacement spectrum is easily constructed from design data and the equivalent-damping estimate is not a problem because, for reinforced concrete, a reasonable amount of nonlinear response (Gulkan, 1974) will justify the use of a damping factor of 10% and response is not sensitive to small changes from 10%. The estimate of the period, however, is not as simple and requires iterations. In the ranges of nearly-constant acceleration and velocity response, response is sensitive to changes in period. Unless the designer has the experience and information to make a confident estimate of the effective period, Algan's approach to estimating drift serves well to help understand the phenomenon of nonlinear drift but its use in design is limited.

A pragmatic approach to determine the nonlinear drift of reinforced concrete structures was provided by Shimazaki (1984). Shimazaki set out in search of an energy criterion to rationalize the use of reduced forces in design. While pursuing his objective, he noticed that, in the range of nearly-constant energy response, he could determine the maximum nonlinear drift of reinforced concrete as a function of the linear drift calculated for a damping factor of 2% of critical. The observation held for a wide range of hysteretic-response types, oscillator-spring strengths, and ground-motion characteristics. It was also confirmed by experimental data. Based on that observation, Shimazaki developed a simple approach to calculation of nonlinear drift of reinforced concrete systems using the following definitions.

DR = Nonlinear-Response Displacement/Linear-Response Displacement
SR = Base Shear Strength/Base Shear for Linear-Response
TR = Characteristic Period/Characteristic Period for Ground Motion

The displacement ratio, DR, normalizes the nonlinear displacement in relation to the displacement that would be calculated for a linear system for the same base motion with a damping factor of 0.02. The strength ratio, SR, expresses the base shear strength of the system as a function of the force that the system would develop if it responded linearly with a damping factor of 0.02. Shimazaki defined the "characteristic period" of the reinforced concrete oscillator as $T\sqrt{2}$ where T is the period calculated for uncracked section. The characteristic period for the ground motion was based on the energy spectrum. It was the period at which the energy response becomes insensitive to increases in period. The practical range of the characteristic period for the ground motion, T_g, is from 0.3 to 1.2 sec except for unusual subgrade conditions.

Using the definitions DR, SR, and TR, Shimazaki concluded that

$$\text{if } TR + SR \geq 1 \tag{1}$$

$$DR < 1 \tag{2}$$

providing a simple procedure to obtain a reasonable bound to spectral displacement for nonlinear systems satisfying Eq. 1. It is important to note the heuristic choices involved in the damping factor and the effective period. Neither was chosen to represent the physical state of the structure. The method does not reflect behavior or cracked-section properties. It provides an answer on the basis of primitive input. Its proof is that it has been shown to work within the domain defined by Eq. 1.

Shimazaki noted that the ratio DR for cases not satisfying Eq. 1 depended critically on the strength ratio, SR, and was beyond the reach of a simple procedure. This was confirmed by the work carried out by Qi (Qi and Moehle, 1991) for that purpose and by Miranda (Miranda and Bertero, 1994) for investigating force-reduction factors.

A study by Lepage (1996) to explore the possibility of eliminating limitation of Eq. 1 ended with an embarrassingly simple answer. The limitation of Eq. 2 could be eliminated by assuming the displacement-response spectrum to be entirely linear. The features of Lepage's solution are described below. Lepage defined T_g specifically as the period at which the energy response spectrum, calculated for a damping factor of 10%, has its maximum. Given T_g, the maximum displacement response is defined by

$$D_{\max} = \frac{F_a * \alpha * g * T_g}{(2\pi)^2} * T \tag{3}$$

with a requirement for threshold strength that rarely governs

$$C_y = \alpha * (1 - TR) \geq \frac{\alpha}{6} \tag{4}$$

D_{max}	=	maximum displacement response
F_a	=	idealized amplification factor for the nearly-constant acceleration range of response for a damping factor of 2% (the factor 15/4 used by Shibata [1976] is a good approximation)
g	=	acceleration of gravity
α	=	ratio of peak ground acceleration to acceleration of gravity
T_g	=	characteristic period of ground motion (if an energy response spectrum is not available this may be taken to be 0.35 for rock or very stiff ground, 0.55 for stiff ground, and 1.2 for soft ground)
T	=	period of vibration
C_y	=	ratio of base shear strength to total weight
TR	=	period ratio, $T\sqrt{2}/T_g$

For a ground motion with the characteristics of El Centro-1940N (T_g = 0.55 sec.) and a peak acceleration of 0.5g, Eq. 3 for spectral displacement can be written simply as

$$D_{max} = 10\,T \tag{5}$$

with D_{max} in in. and T in sec. Barring the likelihood of story mechanisms (concentration of displacement in one story because of a significant weakness in strength of one story with respect to the others), story displacements of a particular structure may be determined from the shape of the linear mode

$$D_i = \gamma * \phi_i * S_d \tag{6}$$

D_i	=	maximum lateral displacement estimate for a given level I
γ	=	participation factor for the mode considered
ϕ_i	=	ordinate for the mode shape considered at level I
S_d	=	spectral displacement from Eq. 5

For low- to medium-rise buildings with reasonably uniform mass, stiffness, and strength distributions, drift is dominated by the first mode. Combination of modal drifts is not justified for design decisions. The change from the first mode is likely to be significantly less than the expected error in estimation of the response spectrum

For a uniform frame, the roof drift is approximately 5/4 times the spectral drift. Using Eq. 5 with the period increased by $\sqrt{2}$, the Mean Drift Ratio (MDR), the ratio of the roof drift to height of roof above base, H in in., would be

$$MDR = \frac{5 * \sqrt{2}}{4 * H}\ 10\ \ * T = 18 * \frac{T}{H} \tag{7}$$

which suggests that for a uniform reinforced concrete frame with a height of 100 ft, a calculated fundamental period T of 1 sec., and the assumed ground motion, the mean drift ratio would be estimated to be 1.5%.

Lepage checked his results based on Eq. 4 using experimental data from 33 multi-story structural models subjected to earthquake-simulation tests (Lepage, 1996). Comparisons of calculated and measured displacements are shown in **Figure 2**.

CONCLUDING REMARKS

One of the reasons deterring the use of drift as a pivotal criterion in design has been attributed to the lack of a simple procedure for estimating it. The procedure discussed above, admittedly no more reliable than the estimate of the ground-motion characteristics, is a very simple one. It does require a displacement response spectrum and the calculation of the period but both of these are well within the designer's reach. The only deterrent that remains is the inertia of habit.

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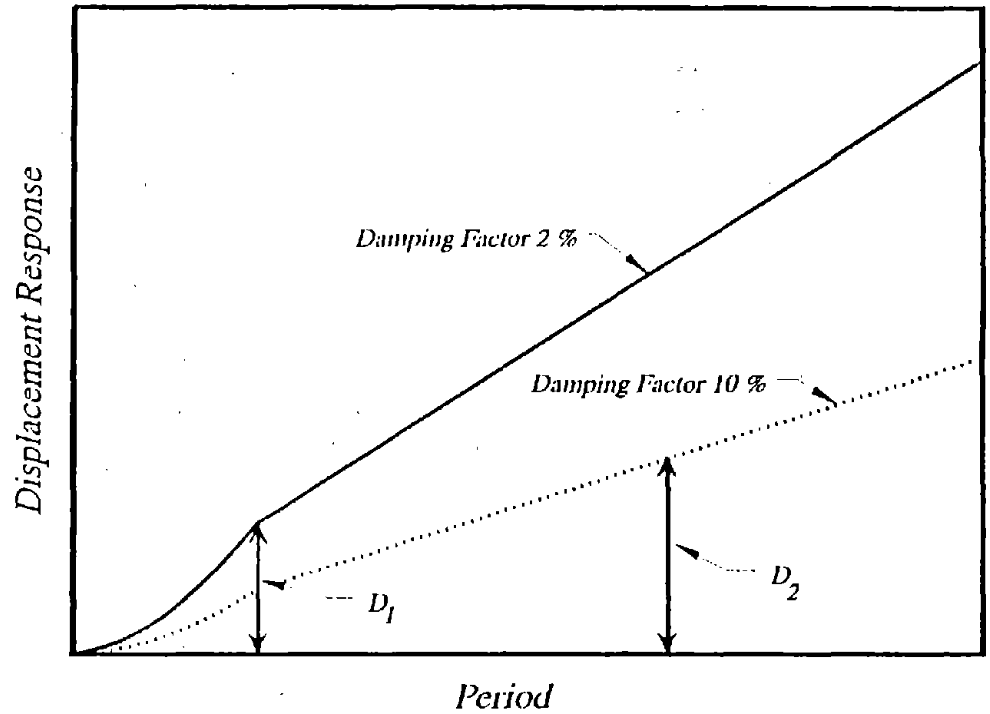

FIGURE 1

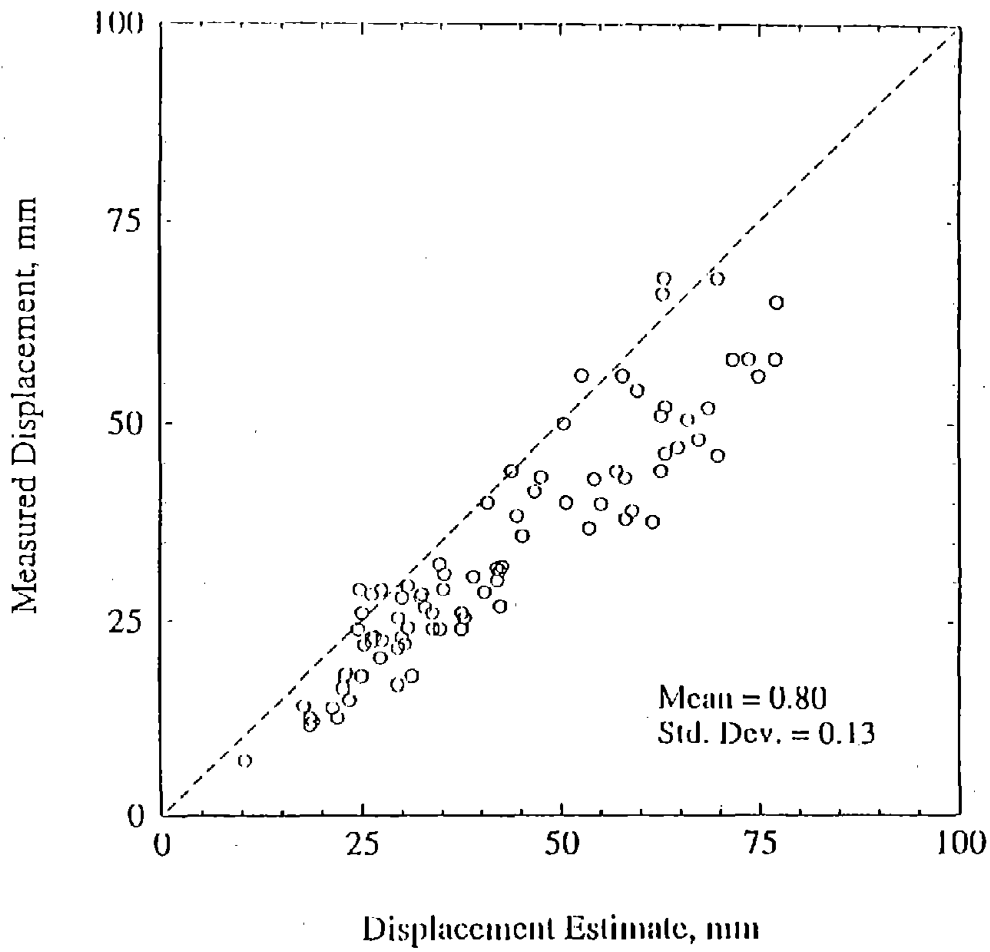

FIGURE 2

New Zealand Code Developments in the Design and Construction of Reinforced Concrete Moment Resisting Frames for Earthquake Resistance

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SUMMARY

New editions of codes for the design of reinforced concrete buildings were introduced in 1995 by the American Concrete Institute and Standards New Zealand. This paper high-lights three areas where differences in seismic design and construction practices for reinforced concrete ductile moment resisting frames exist between the United States and New Zealand, namely the design actions and the quantities of transverse reinforcement in the potential plastic hinge regions of columns, the quantities of transverse reinforcement in the beam-column joints and the anchorage of longitudinal reinforcement passing through interior beam-column joints, and the use of precast concrete in some beam and column elements of ductile moment resisting frames.

INTRODUCTION

New editions of the American Concrete Institute building code for structural concrete ACI 318-95 (American Concrete Institute, 1995) and of the New Zealand concrete design standard NZS 3101:1995 (Standards New Zealand, 1995) were published in 1995. This paper summarises some of the changes made to and the differences between the seismic provisions for ductile reinforced concrete moment resisting frames in those two codes.

DESIGN OF COLUMNS OF DUCTILE FRAMES

DESIGN ACTIONS FOR COLUMNS

The exact characteristics of the earthquake ground motions that may occur at a given site cannot be predicted with certainty and it is difficult to evaluate all aspects of the complete behaviour of a complex structure when subjected to a severe earthquake. Nevertheless it is possible to design a structure so that in the event of a severe earthquake it performs in the most appropriate manner. To ensure that the most suitable mechanism of post-elastic deformation occurs in a structure during a severe earthquake the New Zealand standard for concrete design (Standards New Zealand, 1995) requires that ductile structures be the subject of capacity design. In the capacity design of structures, appropriate regions of the primary lateral earthquake force resisting structural system are chosen and suitably designed and detailed for adequate strength and ductility for a severe earthquake. All other regions of the structural system, and other possible failure modes, are then provided with sufficient strength to ensure that the chosen means for achieving ductility can be maintained throughout the post-elastic deformations that may occur.

For moment resisting frames ductility is best achieved by plastic hinge rotation in selected regions. In tall frames plastic hinges in columns leading to soft stories during severe earthquakes are to be avoided due to the excessive demands on plastic rotations there which has frequently led to collapse. More moderate demands on plastic hinge rotations occur when plastic hinges form only in the beams and at the column bases.

The capacity design rules for protecting columns of tall moment resisting frames, by ensuring that as far as possible strong column-weak beam behaviour occurs, were first introduced in NZS 3101:1982 (Standards Association of New Zealand, 1982) and have remained practically the same in NZS 3101:1995. In general, those rules involve multiplying the column bending moments and shear forces determined from elastic frame analysis, for the load cases involving the ultimate design seismic force, by factors which take into account the beam flexural overstrength, the effects of higher modes of vibration and concurrent seismic forces. The multipliers depend on the frame variables and are at least 1.63. The design axial loads in columns to be used with the amplified column bending moments for the design of the longitudinal reinforcement of column sections should be derived from the shear forces applied at the column faces by the gravity loads on the beams and the moment induced shear forces from the beam plastic hinge moments acting in the two directions concurrently. An adjustment in the moment induced shears is allowed to take into account the probability that not all beam plastic hinges reach their flexural overstrength simultaneously up the height of the frame. A strength reduction factor $\phi = 1.0$ is used when the column design actions are found using this capacity design procedure.

NZS 3101:1995 has only two exceptions to this rule for ductile frames: (1) For one or two storey buildings, or in the top storey of a multistorey building, column sidesway mechanisms are permitted (that is, a strong beam-weak column approach), since the curvature ductility demand at the plastic hinges in the columns in such cases is not high, and (2) If for tall frames strong-column weak beam design is impracticable (for example, if the beams have long spans) some columns may be permitted to form plastic hinges in the top and bottom simultaneously providing that the other columns remain in the elastic range and prevent a soft storey failure. In such case the permitted structure ductility factor used in design may need to be adjusted.

It is to be noted that ACI 318-95 does not require as high a degree of protection of the columns of tall frames as NZS 3101:1995.

TRANSVERSE REINFORCEMENT IN COLUMNS

As a result of recent tests and analytical studies in New Zealand (Watson et al., 1994a) the quantities of transverse reinforcement recommended in NZS 3101:1995 for the confinement of concrete in the potential plastic hinge regions of columns of ductile frames have been made even more dependent on the level of the axial load, resulting in less confining reinforcement in lightly loaded columns and more confining reinforcement required in heavily loaded columns that was recommended in NZS 3101:1982. For lightly loaded columns the requirement for sufficient transverse reinforcement to prevent premature buckling of longitudinal bars is more critical than for confinement of the concrete. The design axial compressive load on columns is not permitted to exceed $0.7N_0$, where N_0 = the concentric load strength of the column. The need for a greater quantity of confining reinforcement at higher axial compressive load levels can be simply demonstrated. The ultimate curvature is expressed by $\phi_u = \epsilon_{cu}/c$, where ϵ_{cu} = ultimate concrete strain and c is the neutral axis depth. As the axial load is increased c will become greater requiring an increase in ϵ_{cu} to achieve the same ultimate curvature ϕ_u. To attain increasingly higher values of $\epsilon_{cu} > 0.004$ requires increasingly greater quantities of confining reinforcement.

ACI 318-95 has retained the same equations for confining reinforcement in columns as were specified in ACI 318-89 in which the quantity of confining reinforcement remains constant for axial loads greater than $0.1 f'_c A_g$, where f'_c = specified concrete compressive cylinder strength and A_g = gross area of column.

An example of the quantities of transverse reinforcement required by NZS 3101:1995 in the potential plastic hinge regions of columns when a curvature ductility factor $\phi_u/\phi_y = 20$ is required is shown in **Figure 1**. Note that the requirement for concrete confinement governs at higher axial loads, and the requirement for preventing premature buckling of longitudinal reinforcement governs at lower axial loads. A comparison of the requirements of NZS 3101:1982 and ACI 318-95 is also shown.

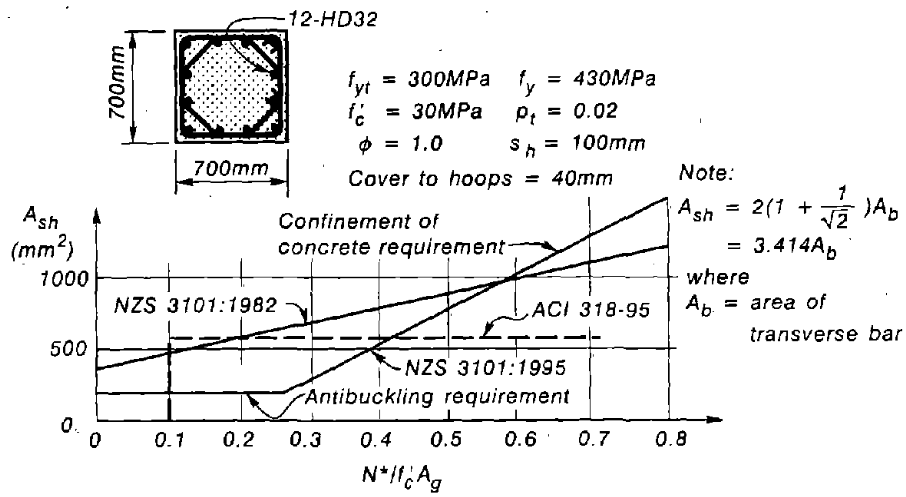

FIGURE 1

EXAMPLE OF TRANSVERSE REINFORCEMENT FOR A DUCTILE COLUMN

For the ductile reinforced concrete moment resisting frames, when the design seismic forces at the ultimate limit state are determined using a displacement ductility factor $\mu = 6$, where strong column-weak beam design is used, NZS 3101:1995 requires design for $\phi_u/\phi_y = 20$ in the potential plastic hinge regions of the bottom storey columns and $\phi_u/\phi_y = 10$ for the potential plastic hinge regions in the columns above the bottom storey. Design for $\phi_u/\phi_y = 20$ is required for the potential plastic hinge regions of the columns of one or two storey frames where strong beam-weak column design is permitted. The transverse reinforcement in columns is placed for the most critical of that required for concrete confinement, prevention of premature buckling of bars and shear resistance.

The confined length of the plastic hinge regions of columns adjacent to the sections of maximum bending moment needs to be sufficiently long to extend over the zone of major plastic curvature and to ensure that the higher flexural strength of the column in the confined region does not lead to flexural failure of the column in the adjacent less confined region. The second requirement is particularly important for columns with high axial compression, since for such columns the flexural strength is markedly increased by confinement of the concrete (Priestley and Park, 1987; Watson and Park, 1994a and 1994b). In NZS 3101:1995 the confined end length of column ℓ_c for low axial load levels when $N^* < 0.25 f_c' A_g$ is taken to be the greater of the column depth h or where the moment exceeds 0.8 of the adjacent end moment, and ℓ_c for high axial load levels with $N > 0.5 f_c' A_g$ is taken to be the greater of 3h or where the moment exceeds 0.6 of the adjacent end moment. An intermediate value of ℓ_c' is taken for axial load levels in between. These values for ℓ_c for high axial load ratios are greater than those specified in ACI 318-95.

BEAM-COLUMN JOINTS

SHEAR REINFORCEMENT

The provisions of NZS 3101:1982 for shear reinforcement in beam column joints of ductile moment resisting frames were of necessity conservative, due to the limited test information available in the late 1970s when those provisions were drafted. In the light of tests and analytical studies conducted in New Zealand (for example: Park and Dai, 1988; Cheung et al., 1991), in NZS 3101:1995 the quantities of shear reinforcement required in joint cores of ductile frames is significantly lower (at least 30% less) than that required by NZS 3101:1982.

The assessment of the shear strength of beam-column joints is based on the contributions of two mechanisms; a diagonal compression concrete strut mechanism transferring the compression forces from the beam and column actions without the aid of shear reinforcement, and the other a truss mechanism transferring bond forces from the longitudinal bars utilising horizontal and vertical joint shear reinforcement and concrete struts. In NZS 3101:1982 it was considered that the truss mechanism was required to carry most of the joint shear when the column axial load was low. In NZS 3101:1995 it is recognised that part of the bond forces from the longitudinal bars passing through the joint core will be transferred by the diagonal compression strut mechanism because of some bond deterioration resulting in some bar forces being transferred directly to the end of the diagonal compression strut. Thus a more significant part of the joint shear can be transferred by the single diagonal compression strut.

The fundamental difference between the NZS 3101:1995 and the ACI 318-95 design approaches for transverse reinforcement in beam-column joints is that whereas NZS 3101 regards that reinforcement as being placed mainly to resist joint shear, ACI 318-95 regards it more as confining reinforcement governed by the quantity placed in the adjacent ends of the columns. Also, NZS 3101:1995 insists on vertical shear reinforcement being present in the joint, normally consisting of intermediate longitudinal column bars placed in the plane of bending between corner longitudinal column bars. This vertical reinforcement also has the function of improving the bond of the longitudinal beam bars by clamping action.

ANCHORAGE OF LONGITUDINAL REINFORCEMENT IN INTERIOR JOINTS

Studies in New Zealand (Park and Dai, 1988 and Cheung et al., 1991) have indicated that a number of factors need to be taken into account when determining anchorage lengths for the longitudinal reinforcement of beams passing through interior beam-column joints. Based on their considerations, the requirement of NZS 3101:1995 for the ratio of the diameter of the longitudinal beam bar to column depth is:

$$\frac{d_b}{h_c} \leq 3.3\,\alpha_f\,\frac{\sqrt{f'_c}}{\alpha_0 f_y} \tag{1}$$

where $\alpha_f = 0.85$ when beam bars pass through a joint in two directions as in two-way frames or 1.0 when bars pass only in one direction, $\alpha_0 = 1.25$ when plastic hinges in beams are developed at column faces or 1.0 when by relocation of plastic hinges in beams the sections at the column faces remain in the elastic range. NZS 3101:1995 also recommends as alternative to Eq. 1 a further equation involving more parameters which gives some relaxation in the d_b/h_c ratio required by Eq. 1. Equations also exist in NZS 3101:1995 for the limiting diameter of vertical column bars passing through beam-column joints.

It is evident that Eq. 1 for interior beam-column joints will generally allow larger d_b/h_c ratios than were permitted by NZS 3101:1982 when high strength concrete is used. For example, for one-way frames with plastic hinges forming in beams adjacent to columns and f_y = 414 MPa (60,000 psi), Eq. 1 gives $d_b/h_c \leq 1/34$ if f'_c = 21 MPa (3,000 psi) and $d_b/h_c \leq 1/22$ if f'_c= 50 MPa (7,300 psi).

ACI 318-95 requires for longitudinal beam bars passing through interior beam-column joints constructed of normal weight concrete of all compressive strengths that $d_b/h_c \leq 1/20$. Hence the possibility of bond deterioration leading to significant bar slip through beam-column joints of ordinary strength concrete is apparently accepted. In New Zealand significant bar slip during a severe earthquake is considered undesirable for two main reasons: (1) It leads to considerable reduction in stiffness of the frame which is residual, and bond deterioration is difficult to repair by epoxy resin injection, and (2) It leads to a reduction in the available curvature ductility factor of the adjacent plastic hinges in the beams. If the bond deterioration is significant the bar tension will penetrate through the joint and the bar tensile force will be anchored in the beam on the far side of the joint. This means that the "compression" steel there will actually be in tension. The result is that, although the flexural strength of the beam may not be greatly reduced, the available ultimate curvature at a specified ultimate concrete compressive strain may be very greatly reduced, for example by 50% (Hakuto et al., 1995).

MOMENT RESISTING REINFORCED CONCRETE FRAMES INCORPORATING PRECAST CONCRETE ELEMENTS

In New Zealand wide use is currently made of precast concrete for some elements of ductile moment resisting frames. The incorporation of precast concrete elements has the advantage of avoiding as far as possible of the fabrication of complex reinforcing details on the building site, high quality control, reduction in formwork and site labour, and increased speed of construction. The cast-in-place reinforced concrete provides the structural continuity necessary for adequate seismic performance. Also, most buildings constructed in New Zealand currently have moment resisting frames as their only lateral force resisting system. Architects prefer the resulting clear areas of floor space within the building when walls are absent and structural symmetry is easily achieved with frames. The general trend is to design the perimeter frames with sufficient stiffness and strength to resist most of the design seismic forces. The more flexible interior columns of the building then carry mainly gravity loading and can be placed with greater spacing between columns. For the perimeter frames the depth of the beams may be large without effecting the clear height between floors inside the building. Also, the columns of the perimeter frames can be at close centres. The use of one-way perimeter frames avoids the complexity of the design of beam-column joints of two-way moment resisting frames.

The seismic design and construction of moment resisting frames incorporating precast concrete elements requires satisfactory methods for connecting the precast concrete elements together. The precast elements are normally connected by reinforcement protruding into regions of cast-in-place reinforced concrete. If the connections between the precast elements are placed in potential plastic hinge regions, the design approach in New Zealand is to ensure that the behaviour of the connection region approaches that of a monolithic cast-in-place concrete structure (monolithic emulation) (Park, 1995). Three common arrangements of precast concrete members and cast-in-place concrete, forming ductile moment resisting multi-storey reinforced concrete frames, commonly used for strong column-weak beam designs in New Zealand, are shown in **Figure 2**. A further commonly used system involving pretensioned precast concrete U-beams and cast-in-place reinforced concrete is also shown in **Figure 2**. All of these systems can be designed for ductile or limited ductility behaviour. This type of construction is little used in the USA.

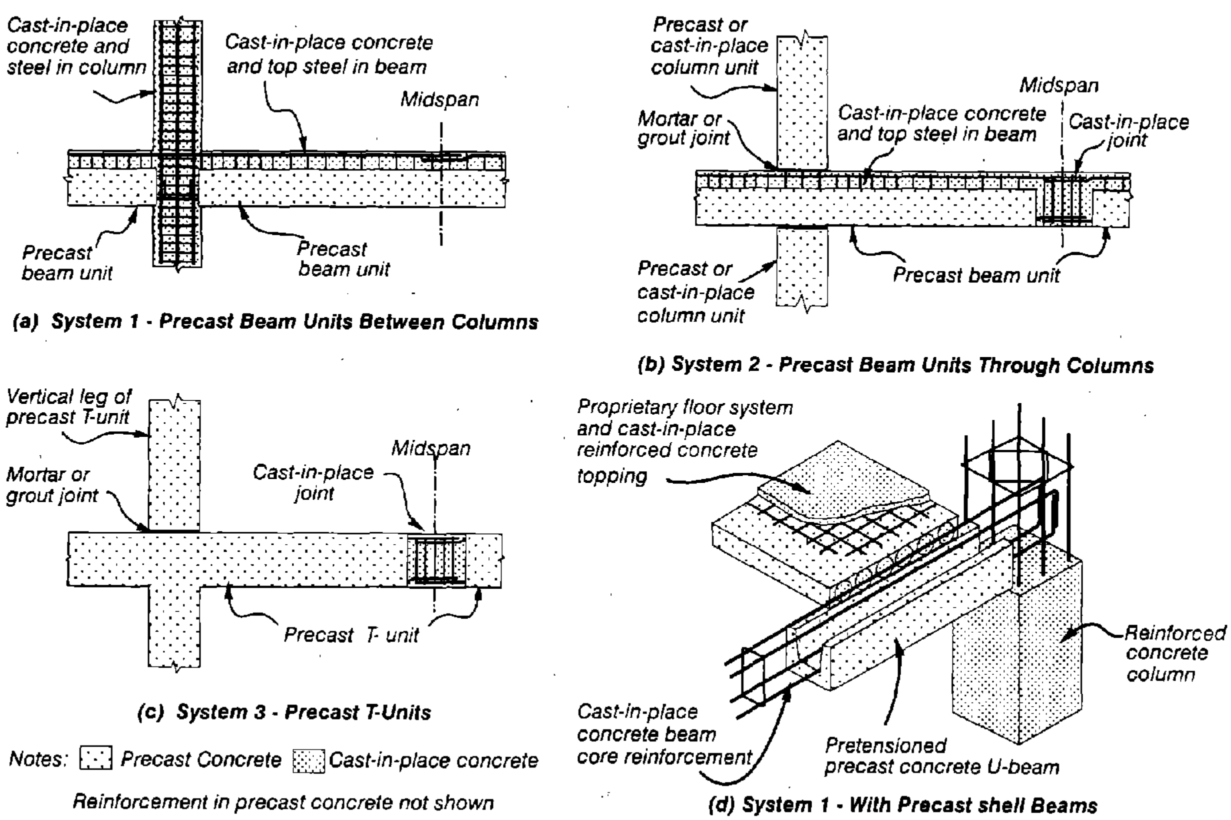

FIGURE 2
ARRANGEMENTS OF PRECAST CONCRETE MEMBERS AND CAST-IN-PLACE CONCRETE FOR CONSTRUCTING MOMENT RESISTING FRAMES

Many of the currently used connection details used in New Zealand have now had experimental verification (Restrepo et al., 1994a; Park and Bull, 1986). The verification involved simulated seismic loading tests conducted on typical beam-column joint specimens to determine the performance of the hooked bar anchorage of the bottom bars of the beam in the cast-in-place reinforced concrete joint core in System 1 of **Figure 2**, the performance of the grouted vertical column bars which pass through vertical ducts in the precast beam in System 2 of **Figure 2**, and the performance of connections incorporating precast prestressed concrete beam shells. Simulated seismic loading tests have also been conducted to determine the performance of cast-in-place reinforced concrete mid-span connections between precast beam elements. The points of interest being the type of spliced connection of longitudinal beam bars (straight splice, hooked splice or diagonal reinforcement) and the distance of the splice from the column face. It was found that behaviour equivalent to totally cast-in-place concrete construction could be achieved by the properly designed beam-column joints and mid-span connections tested. The results of the experimental studies have led to design provisions (Restrepo et al., 1994b) which have been incorporated in NZS 3101:1995.

Commonly in New Zealand floors are constructed from precast concrete units. The floors as well as carrying gravity loads need to transfer the seismic forces to the supporting structures through diaphragm action. Generally diaphragm action is achieved by placing a 65mm (2.6 in) thick cast-in-place reinforced concrete topping slab over the precast units.

CONCLUSIONS

Several changes have been made to the seismic provisions of the recently published New Zealand seismic design standards as a result of the research and development that has been conducted in the 1980s and 1990s. Three of these modifications for ductile moment resisting frames which are high-lighted in this paper are the design actions for columns and the quantities of transverse reinforcement required in the potential plastic hinge regions of columns, the quantities of transverse reinforcement required in beam-column joints and the anchorage of longitudinal reinforcement passing through interior beam-column joints, and the use of precast concrete in some beam and column elements of ductile moment resisting frames. In the case of each of these items differences exist between New Zealand and US practice.

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Impact of High-Strength Material on the Seismic Design of Reinforced Concrete Buildings

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SUMMARY

With the use of high-strength concrete and steel in the seismic design of reinforced concrete buildings, the importance of pre-yield behavior is increased. Structures do not go into post-yield, or perfectly plastic, stage even under the severest earthquake ground motion considered in the design, as shown in the design example of a sixty-storied building. This implies that the pre-yield behavior such as cracking, stiffness reduction due to cracking, pre-yield hysteresis, effect of vertical loads, etc., must be more carefully evaluated than it used to be, and the research effort should also be directed to the same orientation.

INTRODUCTION

It has been traditionally maintained for the seismic design of reinforced concrete frame buildings that the weak-beam and strong-column collapse mechanism should be aimed at and that sufficiently large ductility should be provided to cover the possible post-yield seismic response. With the use of high strength concrete and steel, which enables us to design even higher reinforced concrete buildings in seismic areas, no change is observed in the above-mentioned trend. However looking into the detailed seismic behavior of such buildings, a significant difference becomes apparent. It is the increased importance of pre-yield range in the seismic response.

In the course of "the New RC" national research project of 1988-1993, which was carried out to enable high strength concrete and high strength steel to be used for the design and construction of advanced reinforced concrete buildings in Japan, several design examples were worked out to explore the possibilities for new type of building construction. An example was a sixty-storied apartment building. It shows that, although designed in the same principle of weak-beam strong-column collapse mechanism, not very much plastic flow was observed in seismic response analysis to the severest earthquake ground motion considered in the design. This is mainly due to long natural period associated with the height of the building, and also due to large yield deflection associated with the high strength, hence large yield strain, of steel.

In this paper, a short description on the outline of the New RC Project, and New RC Structural Design Guidelines, will be presented, followed by the design example, static pushover analysis, earthquake response analysis, and evaluation of member ultimate behavior. It will be shown that with the use of high-strength material, the importance of pre-yield behavior is increased. Behavior such as cracking, stiffness reduction due to cracking, pre-yield hysteresis, effect of vertical londs, and so on, must be more carefully evaluated in this new circumstances. More research effort is needed in this direction.

OUTLINE OF THE NEW RC PROJECT

The Building Standard Law in Japan provides structural design basis for buildings up to 60m in height. Structural design, particularly seismic design, of any taller buildings is subjected to the review of the Technical Appraisal Committee for Highrise Buildings of the Building Center of Japan. As far as reinforced concrete (RC) buildings are concerned, the height had been limited to about 20m in practice by the administrative guidance. Any building taller than, say, seven stories had to be constructed by steel structure or composite steel and reinforced concrete (SRC) structure. This administrative guidance was a tradition since 1923 Kanto Earthquake.

Starting around 1980 the situation changed rapidly. There is currently remarkable increase of highrise RC construction. Kajima Construction Co. broke out this movement, by completing the first highrise RC, an 18-story apartment building, in 1974, followed by another 25-story apartment building in 1980. These early highrise RC buildings were realized after long and extensive effort in research and development of the company. Other construction companies followed, and the number of highrise RC buildings increased to more than ten annually in the recent years.

The quick development of highrise RC construction owes to many things, such as large scale structural testing, advanced analysis technique, and development of construction technology. But the most significant factor would be the development of high strength concrete up to 48 MPa and high strength, large size reinforcing bars up to SD390 D41 bars. In an attempt to further promote development of advanced RC construction in the seismic zones, the Ministry of Construction managed a national five year research project from 1988 until 1993. This is called the New RC project, which is a very ambitions project to enlarge the scope of RC construction to a new height in the seismic country such as Japan, probably to 200m or more. The technology developed in this project can be regarded as an attractive new technology to enhance the possibility of RC construction.

The range of material strength set out as the target of this project includes concrete from 30 to 120 MPa and steel from 400 to 1200 MPa. It is obviously unrealistic to assume that behavior of New RC structures can be understood simply by extrapolating the knowledge of ordinary RC structures. Experimental approach was indispensable, but theoretical examination of experimental data was emphasized in this project. Current technical knowledge on RC structures was also re-examined.

For very high strength materials, such as concrete over 60 MPa or steel over 800 MPa, basic problems had to be re-examined, and hence the project did not yield much practical results. Most practical results were obtained for concrete up to 60 MPa and steel up to 700 MPa.

The first major effort was the development of high strength concrete and steel, together with their test method and evaluation criteria. A method to evaluate structural performance of New RC elements and structures was developed, primarily through theoretical studies, which was subsequently investigated experimentally.

New RC Structural Design Guidelines were developed mainly for earthquake resistance. It is based on the dynamic response analysis with a clear definition of required safety. These guidelines will be applicable to RC structures in general, and its philosophy should be applicable to structures of other material. Several design examples were worked out using the New RC materials.

A major achievement in the construction engineering was the development of New RC Standard Specification. It is different from the current JASS (Japan Architectural Standard Specification) in the definition of concrete strength. Concrete strength in the New RC Standard Specification is based on the strength development of concrete in the structure and cylinders under corresponding curing condition, in order to procure the specified strength in the structure with the maximum reliability.

NEW RC STRUCTURAL DESIGN GUIDELINES

FEATURES OF THE NEW RC STRUCTURAL DESIGN GUIDELINES

The New RC Structural Design Guidelines present a structural design method for highrise buildings, but not in a specification style on detailed procedures of structural member proportioning. Rather it aims at basic principles to establish required performance of a building and method to evaluate behavior of a building to be designed.

The design of a structure involves various kinds of external loading. However Japanese RC buildings are usually governed by seismic design considerations. For this reason the proposed guidelines deal mainly with the seismic design. Some specific features of the guidelines are introduced below.

The guidelines introduce seismic safety investigation by means of dynamic and static analyses in three stages, namely, level 1, level 2, and post-level 2. For level 1 earthquake ground motion which would happen once in the lifetime of the building, serviceability should be maintained. For level 2 earthquake ground motion which may be the possible maximum motion to the structure, safety must be maintained. For the post-level 2 stage, the structure should still maintain suitable collapse mechanism and lateral load-carrying capacity.

The guidelines include proposal of earthquake ground motion that should be used in the design of New RC structures. This proposal was made as an attempt to rationalize the currently prevalent use of available strong ground motion records such as El Centro 1940 or Hachinohe 1968. As a part of above-mentioned rationalization, three dimensional earthquake ground motions are considered. Practical application of this consideration is also given.

The safety of a structure under level 1 and 2 earthquake ground motions is specified in the levels of material or members. For the post-level 2 motions, the overall structural safety is to be investigated. Concept of dependable strength and upper bound strength was introduced considering the variation of material strength and accuracy of strength evaluation equations. This simplifies the probability estimation of assumed performance.

Soil-structure interaction and superstructure-substructure interaction are to be considered in the design of foundation and evaluation of earthquake input to the superstructure.

These features are quite general in nature, thus the basic concept of the guidelines is believed to be applicable not only to New RC structures but also to other concrete or steel structures. It is quite natural, of course, to assume that much works would have to be done before such application becomes practical.

EARTHQUAKE RESISTANT DESIGN CRITERIA

As was previously introduced, two levels of intensity are used for design earthquake ground motion. Level 1 ground motion is the largest ground motion expected to occur once during the lifetime of a building, and corresponds to earthquake ground motion of a return period of approximately 100 years. Level 2 ground motion is the largest ground motion that is possible to occur at the site, and corresponds to earthquake ground motion of a return period of approximately 400 years.

For an assumed building lifetime of 100 years, the probability of earthquake intensity exceeding the design level is 60 percent and 20 percent for level 1 and 2 earthquake ground motions, respectively. In general, the intensity of a level 1 ground motion would be approximately 0.4 times the intensity of a level 2 ground motion.

Seismic response of a structure is controlled by the story drift and the structural drift. The story drift is defined as lateral story deflection divided by story height. The structural drift is defined as lateral deflection at the height of the centroid of static lateral forces divided by the height at that level. Three limiting drift levels are identified in the guidelines. They are serviceability drift limit, response drift limit, and design drift limit.

The serviceability drift limit, in terms of story drift, is used to control structural and non-structural damage. The response drift limit, in terms of structural drift, is intended to control the deformation under the possible strongest intensity ground motion. The design drift limit, also in terms of structural drift, is used to examine the deformation at yield hinge regions and to determine the design force level in non-yield hinge regions under the probable largest response deformation considering uncertainties.

The serviceability and response drift limits may be selected by a structural designer, but should not exceed 1/200 and 1/120, respectively. The response drift limit may be determined considering the extent of damage that can be repaired, and significance of the P-δ effect on structural response especially in a highrise building. The design drift limit is defined as a structural drift at which the work done by lateral loads becomes two times that at the response drift limit.

A structure must satisfy serviceability performance criteria for level 1 earthquake ground motions. The serviceability is examined by nonlinear earthquake response analysis. The serviceability criteria are: (1) story drift should be less than the serviceability drift limit, (2) no structural members should, in principle, develop yielding and (3) nonstructural elements should not be damaged.

A structure must satisfy safety performance criteria for level 2 earthquake ground motions. Safety criteria were prepared for the nonlinear earthquake response analysis and for the nonlinear static analysis (pushover analysis) separately. The static analysis is required to compensate the uncertainty in the characteristics of earthquake ground motions, the reliability of analytical methods, and a limited number of ground motions used in the response analysis. The response analysis is required to take into account the dynamic effect, that is, the effect of force distribution under earthquake excitation different from the assumed static force distribution.

The safety criteria for the response analysis are: (1) maximum structural drift should be less than the response drift limit, (2) maximum story drift should be less than 1.5 times the maximum structural drift, (3) the yield hinges must maintain its full resistance, (4) the location where yielding is not permitted should not develop yielding, and (5) brittle failure, such as shear failure or bond splitting failure, should not take place in any member.

The safety criteria for the pushover analysis up to the design drift limit are: (1) the yield hinges must maintain its full resistance, (2) the location where yielding is not permitted should not develop yielding, (3) brittle failure should not take place in any member, and (4) the lateral resistance in terms of base shear coefficient should not be less than 0.25 RtZ at the design drift limit. The required strength of New RC building is about the same as that of recently constructed highrise buildings

DESIGN EARTHQUAKE GROUND MOTION

The design earthquake ground motion is directly used in the response analysis for levels 1 and 2 criteria checking., hence, it is of utmost importance for the design of New RC buildings. Design earthquake motions should be determined considering seismicity of the site and ground conditions. The guidelines propose the spectral characteristics for level 2 ground motion covering period range up to 8 seconds, and it is recommended that the simulated ground motions developed from this spectrum should be used simultaneously with the currently used strong motion records. The level 1 ground motion is assumed to be 40 percent of the level 2 motion. This was derived from the study concerning the return periods of two levels of earthquake ground motion.

The bidirectional horizontal earthquake motions develop varying axial force in a corner column significantly larger than a uniaxial ground motion due to the bidirectional overturning effect, and also develop simultaneous bidirectional bending moments and shears in the column. The guidelines require that the safety of a structure should be examined for uniaxial horizontal ground motions and uniaxial horizontal static forces, but occurring in all possible directions. The earthquake motion or static horizontal forces in the orthogonal direction is ignored; small additional design forces are shown to be sufficient to cover the orthogonal effect.

MODELING OF STRUCTURE

In the practical design, it is convenient to analyze a building by different models according to the different methods of analysis. Each model is idealized in a way that the objective of the particular analysis can be achieved. The static pushover analysis should be carried out based on an appropriate frame model, preferably a space frame model, taking into account the nonlinear mechanical properties of constituent members. The dynamic analysis is performed basically in order to investigate the drift during earthquake excitation. Hence it is not always necessary to be done by a frame model. Provided that the nonlinear properties of members are adequately reflected, simple mass-spring model may be used.

The guidelines suggest to use a probability of non-exceedance of 0.90 in determining the dependable and the upper bound resistance on a statistical basis of experimental data. The dependable resistance must be used in all members, when response drift of a structure is examined in the earthquake response analysis under level 1 and 2 earthquake ground motions, and when lateral force resisting capacity of a structure available at the design drift limit is examined in the static analysis. The upper bound resistance is used at the location of allowed yield hinges in the static analysis when design actions are determined for a region other than allowed yield hinge regions or when the brittle failure of a member is examined.

The stiffness of an RC member may be assumed to change at cracking and yielding. A yield point in the guidelines is defined as the point at which the stiffness degrades significantly under monotonically increasing force. The yield resistance may be estimated by routine procedure for the tensile yielding of longitudinal reinforcement.

The guidelines suggest to use average values for the initial stiffness, cracking moment and yield deformation whereas the dependable and upper bound yield resistance is used to take into account the variation of material strength and the reliability of evaluation methods. The yield deflection of columns does not change appreciably with yield resistance for a wide variety of parameters, while the yield deflection of girders increases with yield resistance.

Hysteretic characteristics may be assumed using routinely used nonlinear models, but its parameters must be properly selected to account for the hysteretic energy dissipation of members.

NEW RC DESIGN EXAMPLE

OUTLINE OF THE BUILDING

In the course of the New RC project, several design examples were worked out to investigate the potential of the New RC material to build super-highrise buildings in the seismic area. Here is presented a design example of 60-story apartment building.

The typical floor of the building consists of six-bay frames of 5.7m span in two directions. No shear walls are provided above ground level. The total width of the plan is 37.2m in each direction including balcony cantilever slabs. The height of the building above ground is 175.6m, and the typical story height is 2.9m. The aspect ratio of the building is 5.1, being a considerably slender structure. The building has three levels of basement, consisting of eight-bay frames in two directions. These basement levels are assumed to be very strong and stiff with a rich amount of shear walls and retaining walls. The building has a raft foundation which rests on a hard gravel soil layer.

Concrete up to 60 MPa, axial reinforcement of USD 685B, and lateral reinforcement of USD 785 are used. Floor slabs have thickness of 165mm. Yield hinges are expected to occur at all girder ends and first story column bases, but it was deemed desirable to have no yield hinges at the first story column bases for the frame stability up to the deflection corresponding to the response drift limit.

Column sections vary from 1m square at the first story to 75cm square at 41st story and above. Exterior columns below 20th story are provided with "center bars" in addition to ordinary peripheral column bars, to account for the excessive axial forces due to overturning moment. Girder sections vary from 45cm by 90cm at the second floor to 40cm by 70cm at the 42nd floor and above. Four top and four bottom bars are the typical axial reinforcement, with two each of additional bars in the second layer for girders in the lower floors.

STATIC PUSHOVER ANALYSIS

Using a three-dimensional static nonlinear frame analysis program, pushover analysis was conducted in the frame direction as well as diagonal direction. Dependable strength analysis and upper bound strength analysis were carried out separately. Lateral force distribution determined from the SRSS method for proposed response spectrum was used. The centroid of lateral force is located at the 39th floor.

The results of pushover analysis in the frame direction showed that virtually no yield hinges formed at the design drift limit even in case of dependable strength. Two analyses based on dependable and upper bound strength become essentially the same in this case. Story drift distribution was fairly uniform up to the design drift limit because the frame remained in the preyield stage. At the response drift limit of 1/140, the base shear was 0.227 RtZ. At the design drift limit of 1/90, the base shear was 0.312 RtZ, which was 1.25 times the required value of 0.25 RtZ. A few girders in the exterior frames formed yield hinges at this stage, but no yielding was seen in the interior frames. The results of analysis in the diagonal direction were quite similar to above. At the response drift limit of 1/140, the base shear coefficient was 0.252 RtZ. At the design drift limit of 1/90, the base shear was 0.335 RtZ, which was 1.34 times the required 0.25 RtZ.

EARTHQUAKE RESPONSE ANALYSIS

Two types of analytical models were adopted for the earthquake response analysis. One was the mass-spring model, whose nonlinear characteristics were adopted from the static pushover analysis. As mentioned above, very few yield hinges formed at the design drift limit, and so analysis was continued to form trilinear skeleton curves. Six input ground motions were used, and analysis was performed both in the frame as well as diagonal direction. Another analytical model for the earthquake response analysis was the three-dimensional frame model. An input ground motion which produced the largest response in the mass-spring analysis was used for the frame analysis.

The results of mass-spring analysis were as follows. Under level 1 ground motion in the frame direction, maximum story drift was only 1/260, or 1.11cm, at the 9th story. Under level 2 ground motion in the frame direction, maximum structural drift was 1/156, and maximum story drift was 1/123, or 2.36cm, at the 23rd story. The level 2 motion in the diagonal direction resulted in smaller response in terms of structural drift of 1/163, but maximum story drift was larger, being 1/114 at the 7th story.

The results of frame analysis showed no yielding to occur under level 2 ground motion. The maximum member ductility factor was 0.95 at some external girder ends of exterior frames, but the ductility factor at the first story column base was less than 0.3.

CONCLUSION

The main purpose of conducting example design in the New RC project was to demonstrate the potential of high strength material in the highrise and other advanced structures in seismic zones. The design example introduced here shows that an apartment building of 60 stories can be constructed in seismic zones using concrete up to 60 MPa and steel up to 685 MPa for axial reinforcement and 785 MPa for lateral reinforcement. Span length of 5.7m and column size of 1m in lower stories may be disappointing for architectural planning. However with the use of even higher strength materials, it will be possible to make span length and column size more realistic.

An important biproduct of this design example was that, even in case of level 2 earthquake ground motion, virtually no yield hinges form in any part of the structure. Although cracking was indispensable in most girder ends, steel behave essentially in the elastic range. In the past there is a trend to emphasize post-yield behavior, and at the same time to de-emphasize cracking, pre-yield hysteresis, effect of vertical load, and so on. It is necessary that these pre-yield behaviors are studied more carefully, so that the analysis becomes more accurate and dependable.

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The Role of Observation in Structural Engineering

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SUMMARY

The state-of-the-art of structural engineering has been advanced immeasurably through the observations made by engineers of buildings in service and through physical tests of structural components or of entire structures. With the introduction of computer technology into the practice of structural engineering, there appears to be increasing reliance on computed results and less reliance on visual observation and physical testing. The role of government, industrial, and academic policies have also played an important role in changing attitudes and professional practice. Over his career. no one has been a more passionate advocate for experimental and field studies in structural engineering than Professor V. V. Bertero.

The purpose of this paper is to review several studies that Prof. Bertero carried out and to recall some of the observations he made regarding their importance and their role in understanding structural behavior. The current status of experimental research in the United States is reviewed briefly and some comments regarding the need and opportunities for changing direction are offered.

BACKGROUND

In 1977, Bertero was the organizer of a workshop on Earthquake-Resistant Reinforced Concrete Building Construction (ERCBC) sponsored by the National Science Foundation. The workshop was intended 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." A three-volume set of Proceedings (Bertero, 1978) included the papers presented, the results of discussions, and recommendations, particularly the identification of high-priority needs for advancing earthquake-resistant reinforced concrete construction.

The organizing committee summarized the recommendations that they felt deserved special mention.

1. Improved cooperation and communication between researchers and practicing engineers, among researchers, and on a national and international basis.
2. Dissemination of technical information in simple, comprehensible terms for rapid implementation of findings into practice and into the literature.
3. Integrated analytical and experimental research on three-dimensional linear-elastic and hysteretic behavior of *real* buildings and subassemblages. A number of topics were identified ranging from material studies to foundation effects on building response. Specific topics

mentioned were: effects of bond deterioration, joint flexibility, and floor diaphragm deformability; column behavior under combinations of lateral and axial forces; and effects of non-structural components. Special mention was made of the need to address problems associated with existing buildings including evaluation of the seismic behavior of a variety of typical structural systems, determination of acceptable levels of damage, and examination of techniques for repair and retrofitting.

It is interesting to note that high-priority needs included the need to develop structural floor-wall reaction systems for carrying out these studies, and to make use of existing earthquake simulators to better understand the seismic response of reinforced concrete structures.

The ERCBC Workshop was organized in the years immediately following the 1971 San Fernando earthquake. The structural engineering profession realized the need for changes in seismic design requirements and there was general consensus among researchers, practitioners, and policy makers that research was needed to provide the technical knowledge on which changes in seismic design procedures would be based. The National Earthquake Hazards Reduction Program reflected a national interest in earthquake-related research and activity. In the late 1970's, the United States and Japan entered into a long-term cooperative research program which was centered around the new structural testing capabilities that were being built in Japan. The large-scale structural testing laboratory permitted the construction of a full-scale seven-story reinforced concrete structure in which the primary lateral force-resisting system was a shear wall [Wight (1985)]. The US/Japan program reflected, in large measure, many of the needs that were identified in the ERCBC Workshop--international cooperation, research and practice working together to design tests and to evaluate the results, and the opportunity to study in various scales a structure and its critical components.

LESSONS FROM A FULL-SCALE TEST STRUCTURE

The first structure tested was a combination of US and Japanese practice and it did not truly reflect typical practice in either country. However, it did allow the research teams to examine three key aspects of structural engineering research:

1. Develop a program in which analytical and experimental research was integrated.
2. Evaluate the desirability of testing structures at small scale and defining the limitations of such tests.
3. Examine the behavior of a portion of a structure as part of a whole system versus its behavior when tested as an isolated elements or subassemblage.

Bertero and others [Charney and Bertero (1982), Bertero, et al, (1985)] conducted analytical studies of the structure as designed. The results of the non-linear analyses were then compared with test results. The analyses, which were considered to be the most sophisticated available, underestimated the lateral strength of the structure by 20 to 50%. As a result, there was a considerable effort made to determine why such discrepancy occurred. Bertero's keen insight into structural behavior resulted in explaining the reasons. First, there were walls at the ends of the structure included to provide stability to the system in the direction perpendicular to the applied lateral force (or plane of the structural wall). Although they were not considered to provide any resistance in the direction of lateral force, they provided moment resistance at the exterior or end supports of the frame. The strength of the joints at the beam-column connections was evaluated ignoring the contribution of floor slabs to beam strength. Tests of components at several US and Japanese laboratories indicated that the floor slabs increased flexural strength of the beams [Joglekar et al (1985), French and Moehle (1991)]. This finding could have important repercussions in the case of a design where hinges are expected to develop in the beams and are intended to "protect" the columns from reaching flexural hinging or shear capacity.

However, a major reason for the difference in computed and measured lateral capacity was only apparent because a three-dimensional structure was tested. The flexural capacity of the wall was determined considering only the dead loads acting as a compressive force on the wall section. Bertero et al (1985) observed that as the wall deformed laterally, the edge of the wall in tension tended to lift from the base or foundation while the compression edge did not deflect at the base. As a result of the uplift, the ends of the floor beams (both in the transverse and longitudinal direction with respect to the wall) framing into the uplifting edge also deflected upward. The flexural resistance of the beams developed a "hold down" force on the wall. If these hold-down forces are added to the dead loads to determine the axial force on the wall, the resulting increase in wall moment capacity makes up a significant fraction of the difference between computed and measured lateral strength.

Once the differences between assumptions made in analysis and those required to better represent observed behavior were identified, the changes needed may seem obvious. The cause and effect relationship between the assumptions and the difficulty are seldom surprising, but the role of observation is clearly demonstrated. Without the test of the three-dimensional, seven-story structure, and in large scale, it is unlikely that the differences would have been explained, let alone realized.

EXPERIMENTAL RESEARCH TODAY

In the nearly twenty years that have passed since the ERCBC Workshop, some notable progress has been made in structural engineering as a result of experimental studies. Some of the problems that were identified in 1977 have been the subject of extensive experimental work. The design of joints in moment-resisting frames is now fairly routine; designers understand the problem and guidelines for detailing have been developed [Jirsa (1991)]. One of the success stories in structural engineering has been the improvement in ductility of reinforced concrete structures through improved detailing requirements.

Another that has received considerable attention is seismic rehabilitation. Coordinated national programs have been supported to provide data for evaluating existing structures and for determining the performance of various rehabilitation techniques [Jirsa (1996)]. However, some of the problems identified at the ERCBC Workshop have not received much attention. The performance of floor diaphragms has not been studied in the laboratory. Field observations following the 1994 Northridge earthquake indicated that those elements were critical in distributing and transferring forces to the vertical lateral load resisting system and design requirements for detailing diaphragms may not be adequate.

The need for integrated experimental and analytical studies was reiterated in field observations following the Northridge and 1995 Kobe earthquakes. Both events occurred on faults under heavily developed urban areas. The effects of near-fault ground motions on structures is not well understood and will require a considerable effort to calibrate analytical results using measured ground motions with field observations of buildings that have been subjected to the same or similar ground effects.

The Kobe earthquake also demonstrated that the extrapolation of design equations based on relatively small test specimens to large cross sections needs verification. However, from field observations of failures of large column sections, the profession should appreciate that there are uncertainties in current, widely-used design specifications.

Both of the examples noted above demonstrate the importance of reconnaissance following earthquakes and other disastrous events. In the field, the structure is subjected to real loads and effects of scale and the multi-dimensional nature of structures is always included. The difficulty is in interpretation of the observations. We have not taken full advantage of those observations in improving our knowledge of structural response. While data are available for a large number of structures that were located in the area affected by the Northridge earthquake, relatively few studies have been initiated to use that data for calibration of analytical procedures and for establishing procedures for rapid dissemination of data from that and future events so that the benefits to be derived from such information are optimized.

Over the past twenty years, many new materials and techniques have been utilized in the reinforced concrete construction. In many cases, there has been little verification of the structural behavior of these innovations, nor has there been validation of assumptions made regarding long-term durability or stability of new materials. The widespread use of epoxy-coating is an example of the introduction of a new material that appeared to solve the corrosion problem in reinforced concrete structures, but created other problems that were not well-understood until problems were observed in the field and laboratory studies were undertaken to resolve those problems.

Examples of needs for better understanding structural performance could be cited for areas other than reinforced concrete, such as the use of composites to improve durability of structures and problems with welding procedures and steel materials. As the age of our built environment has increased, there has been growing concern about the condition of the inventory of existing structures and the need for upgrading and improving infrastructure vital to the nation's economy. The need for extending and maximizing the design life of structures has led many to conclude that new approaches for structural design must be developed. Discussions in the structural engineering community regarding performance-based or performance-sensitive design procedures are an indication of that concern. There is a clear need for experimental research to support technological advances in these areas but it has not been coupled with a commitment to carry out such research.

IMPEDIMENTS TO EXPERIMENTAL RESEARCH

The amount of research conducted in the US to support structural engineering is a minuscule fraction of the value of annual construction costs. This is not a new phenomenon but it should be of concern to the profession. The consequences of inadequate performance of structures will, at the very least, create inconveniences to the public and, in a worst case, have a depressing impact on regional or national economies. A bridge out of service for maintenance may be a minor irritation that results in traffic delays or detours but the loss of transportation, industrial, or commercial facilities in an earthquake, such as the event in Kobe, will have lasting impact and may result in permanent changes for the community. A number of reasons may be cited to explain why experimental and field research has languished.

<u>Cost</u> A prime factor is cost and time required to complete experimental research. It is clearly not as expensive to conduct numerical tests. However, analytical procedures must be validated through calibration with experimental and field observation. Likewise, the value of analysis lies in the ability to study parametric changes and to identify areas where additional study will be needed without having to conduct physical tests. As industrial and governmental sponsors of research become concerned with rapid implementation of results and accountability, the amount of funding provided on an annual basis and the time allowed to conduct a study have diminished. Sponsors are unwilling to make long-term commitments that may not have a short-term impact on profits or demonstrate immediate benefits. In many agencies and corporate entities, technical expertise and in-house research capabilities have been "down-sized" or eliminated and managers may change frequently. In addition, much (or all) of the management team may have non-technical backgrounds. Unfortunately, funding for generating technical data and for providing technical input to the decision-making process has not been replaced by input from external technical sources or experts. Technical input may have simply disappeared.

<u>Policy</u> The structural engineering profession has not made a strong enough case for a national policy or program to support experimental and field research. Over the past 30 years, the National Research Council, the Earthquake Engineering Research Institute, and other professional groups have repeatedly stated the need for improving and expanding our experimental facilities [Housner (1982), Abrams (1995)]. However, there has not been any major funding for carrying out such improvements. Professional societies must continue to develop and promote the case for experimental and field research and demonstrate the connection between public safety and observation of structural performance. The public generally reacts to unfavorable publicity--structural failures and personal injuries or deaths. The publicity surrounding earthquake events provides an avenue for explaining the value of observation--those opportunities must not be wasted. When the injuries or loss of life are minor, the interest may not be as great but the public, and certainly enlightened owners, are beginning to understand that loss of life is not the only consequence of poor structural performance.

<u>Academia</u> Much of the experimental capability in the US now resides at academic institutions. While many universities have experimental facilities, the manpower to utilize those facilities is lacking. There must be a critical mass of faculty and technical staff to support a viable

experimental facility. Many institutions have developed experimental facilities and expected research programs to attract capable faculty members. However, experience has shown that a the personnel must be in place for such programs to prosper and that facilities develop based on faculty demands and interests. However, academia has also created an evaluation and promotion process that often mitigates against experimentalists. Since the time to design, construct, test, evaluate, and report large-scale structural engineering research is long and the tenure "clock" does not take the nature of research into account, many outstanding young researchers have decided that their future advancement will be hindered if they "waste" time conducting experiments. Senior faculty members must be sensitive to this problem and support those that have an interest in and a talent for experimental studies. Such support must include mentoring junior faculty as well as "educating" administrators in the allowances that must be made when evaluating faculty performance in various technical areas.

Litigation One of the impediments to making observations of failures in the field is the desire of parties involved in the ownership, construction, design, or operation of facilities to limit information in case litigation will be involved. While that is an understandable reaction, it places severe barriers to the profession in trying to understand the reasons for such failures and to use the information to improve design specifications, construction practice, or operation and maintenance procedures. The legal profession and the insurance industry will need to work with structural engineers in the interests of public interest to make it easier for these unfortunate events to be utilized to make long-term improvements in the safety and design life of the built environment.

Cooperation and Collaboration Although nearly everyone agrees that cooperation and collaboration in professional activities is necessary and valuable, actions are not always consistent with sentiments. Engineers outside academia must also be supportive of researchers efforts in encouraging technical publication of results, working with faculty to develop results that are readily implemented, and letting sponsors of research know the benefits of experimental studies relative to the costs of that research. Building officials, lawyers, policy makers, contractors, materials suppliers, lenders, and insurers must work with engineers to formulate and support research that will benefit agendas the public. Perhaps it is even more important to let the public know the costs of not conducting such research. Engineers from different nations must work together and share findings and facilities. The costs of research to solve many problems are simply too great to be borne by one country or for research to be duplicated. The demands for public safety do not change across national boundaries. It is especially important that the structural engineering profession be united in its priorities and requests for public support. Such priorities must be developed through serious discourse within professional groups and societies. The successes of other scientific and technical organizations and professional groups should be studied and their approaches to reaching consensus in establishing funding priorities emulated.

WHERE TO FROM HERE?

The future of structural engineering depends on developing experts who understand the response of structural systems and are trained to make observations that will lead to improved knowledge and improved designs. As a distinguished faulty member of the University of California at Berkeley, Professor Bertero has been a leader in providing such training to his students and to the profession. He has been widely recognized through his publications and has received many awards here and abroad. The recognition he received as Engineering News-Record's "Man of the Year" was a unique honor not afforded many in research or in academia. If we wish to honor his career in engineering and address the concerns that he has so often expressed, we can do no better than to renew our efforts to improve and expand our national capabilities in experimental research and to resolve to improve cooperation and collaboration between researchers and practitioners, industry and academia, and across institutional and international boundaries.

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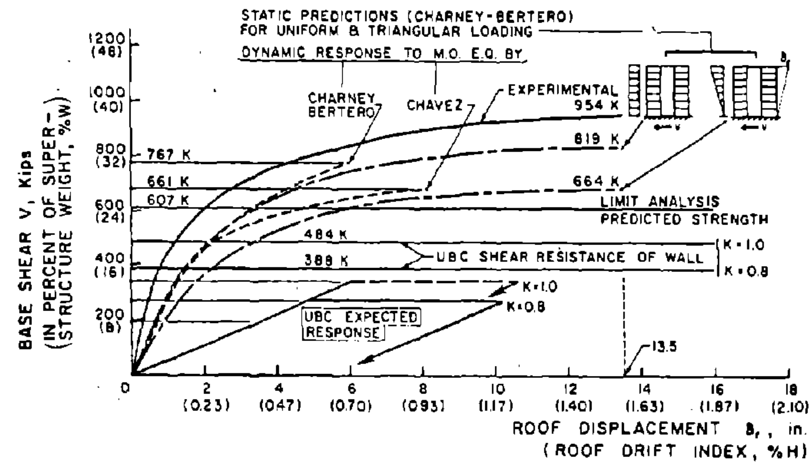
FIGURES

Fig. 1 Comparison of measured and computed lateral capacity, seven-story structure, Charney and Bertero (1982)



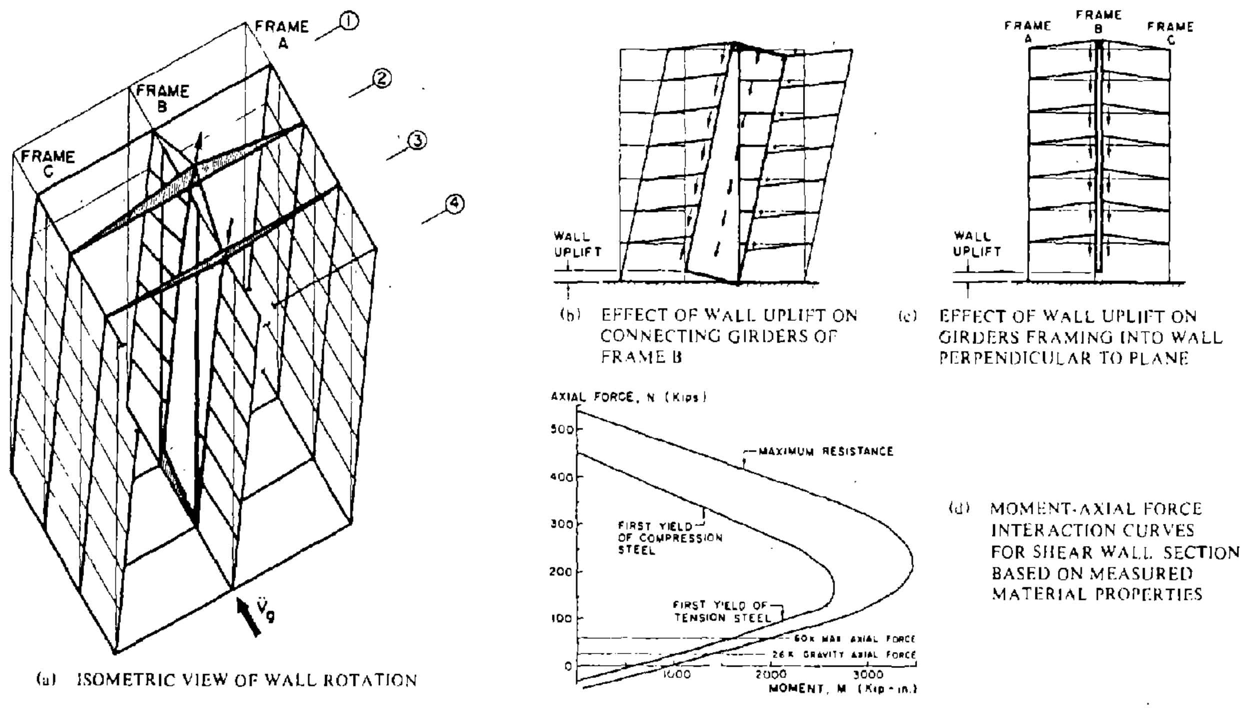


Fig. 2 Effect of "outriggering" action of frames on wall lateral capacity, Bertero et al (1985)

Performance-Based Design of Masonry Structures for Seismic Loads

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ABSTRACT

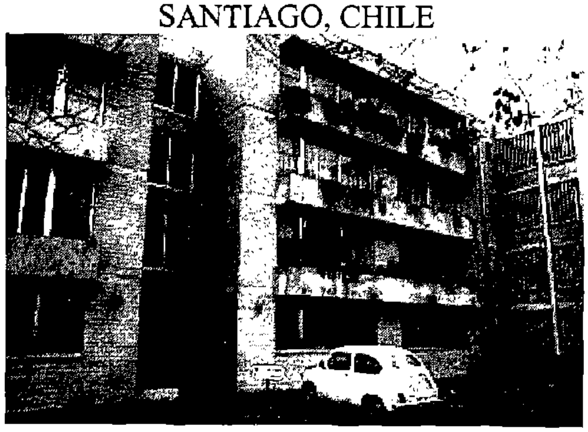
In this paper, recent experience regarding the seismic performance of modern masonry is summarized. The technical basis for that performance is reviewed, particularly the results from the US TCCMAR program. Finally, masonry code updating actions in the US are reviewed, with emphasis on the limit-state MSJC code now being developed. It is concluded that through appropriate control of architectural layout, structural design, material specification, and construction quality, modern masonry is capable of good performance under seismic loads.

RECENT EXPERIENCE WITH THE SEISMIC PERFORMANCE OF MODERN MASONRY

Recent years have offered many examples of the seismic performance of modern masonry:

- the Chilean earthquake of 1985;
- the Loma Prieta earthquake of 1989; and
- the Northridge earthquake of 1994.

FIGURE 1
DUBLÉ ALMEYDA BUILDING, SANTIAGO, CHILE



CHILEAN EARTHQUAKE (1985)

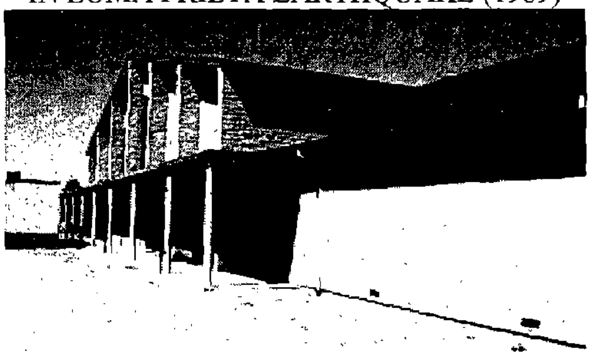
For example, Figure 1 shows the Dublé Almeyda Building, a 4-story apartment in Santiago, Chile that suffered only one small crack. This building is of particular interest for the US, because it had been designed and constructed essentially according to the Uniform Building Code. The performance of this reinforced and grouted concrete masonry building is described in Villablanca (1988, 1989) and Klingner (1990).

LOMA PRIETA EARTHQUAKE (1989)

In the Loma Prieta earthquake, many older buildings of unreinforced masonry (URM) suffered severe damage (Figure 2).

FIGURE 2
OLD URM BUILDING, TOTALLY DESTROYED BY LOMA PRIETA EARTHQUAKE (1989)

FIGURE 3
SCHOOL BUILDING OF MODERN MASONRY, ONLY SLIGHTLY DAMAGED IN LOMA PRIETA EARTHQUAKE (1989)



Nevertheless, the generally excellent performance of modern masonry is exemplified by the school building shown in Figure 3, located less than 15 km from the epicenter, and less than 0.5 km from the causative fault. After the earthquake, it served as an emergency shelter for nearby residents whose houses had been destroyed (Jalil 1993).

NORTHRIDGE EARTHQUAKE (1994)

With respect to earthquake resistance, the Northridge earthquake showed some old lessons that had been learned satisfactorily, other old lessons that had not yet been learned, and a few new lessons.

FIGURE 4
URM BUILDING IN SANTA MONICA (1994)

Some masonry chimneys were damaged, particularly those with improper construction details. Some URM buildings, such as the one shown in Figure 4, were badly damaged. In the early 1980's, Los Angeles adopted a basic retrofitting ordinance for URM buildings ("Division 88"), which required parapet braces, mechanical connections between walls and horizontal diaphragms, and bracing to limit the out-of-plane slenderness of walls. The building shown in Figure 4 was located in Santa Monica, where URM retrofitting was not required.

FIGURE 5
17-STORY HOTEL, UNDAMAGED IN NORTHRIDGE QUAKE

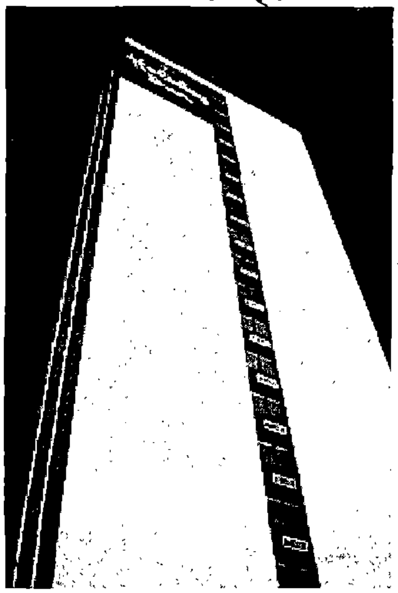
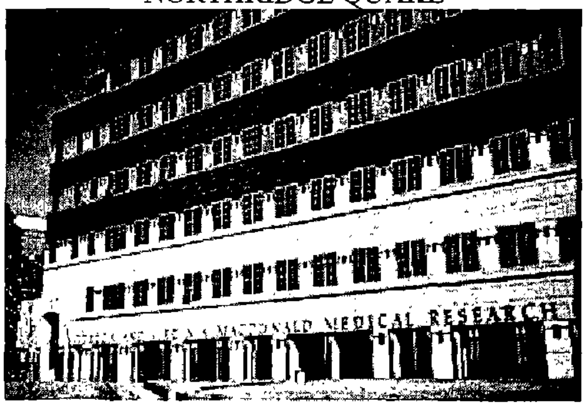


FIGURE 6
MASONRY VENEER, UNDAMAGED IN NORTHRIDGE QUAKE



In contrast, Figures 5 and 6 show examples of the generally good performance of modern grouted and reinforced masonry, and of masonry veneer.

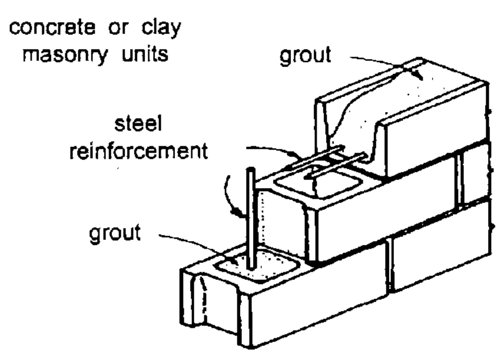
FIGURE 7
CORNER DETAIL OF REINFORCED MASONRY SHOWING CONGESTION

However, isolated problems were apparent. For example, Figure 7 shows damage to a modern reinforced masonry building due to congestion of flexural reinforcement. These and many other examples of masonry performance are discussed in Klingner (1994).

EXPERIMENTAL BASIS FOR SEISMIC PERFORMANCE OF MODERN MASONRY

Most modern US masonry construction involves hollow concrete or clay masonry units, reinforced horizontally and vertically, and filled with grout (Figure 8).

FIGURE 8
TYPICAL US REINFORCED MASONRY

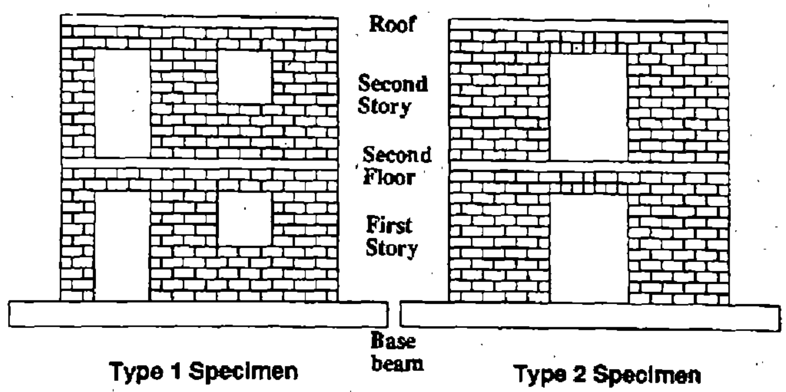


Of particular interest with respect to modern masonry research in the US is the NSF-sponsored "TCCMAR" program (1985-1995), a coordinated program of university and industry researchers whose overall objective was to develop the basis for a limit-state masonry design code (Noland 1990). The program studied basic material behavior, subassemblage behavior, and analytical techniques.

EXAMPLE TCCMAR STUDY INVOLVING WALLS WITH OPENINGS

This study, identified as Phase 3.1(c) of the TCCMAR program, dealt with the in-plane seismic resistance of two-story masonry walls with openings. The overall objectives of this study were to examine how the in-plane seismic behavior of masonry walls was affected by the floor system, and by the openings. As shown in Figure 9, two types of walls were examined, using a total of six specimens.

FIGURE 9
ELEVATIONS OF 2-STORY WALLS WITH OPENINGS, TESTED IN TCCMAR PROGRAM



The specimens were constructed of grouted and reinforced concrete masonry walls, with floor systems of precast planks covered by cast-in-place topping. The specimens were loaded at each story level by equal shears, representing the inelastic distribution of story shears. They were loaded vertically by constant axial loads representing the effect of dead loads from upper stories.

Some specimens were designed to form flexural mechanisms of the column type, as shown in Figure 10. Those specimens had relatively small quantities of flexural reinforcement at the tops and bottoms of first-floor piers, where the flexural hinges were intended to form. Other specimens were designed to form flexural mechanisms of the coupled-wall type, as shown in Figure 11. Those specimens had relatively small quantities of flexural reinforcement at the wall base, where the principal flexural hinge was intended to form.

FIGURE 10
PIER-TYPE COLLAPSE MECHANISM

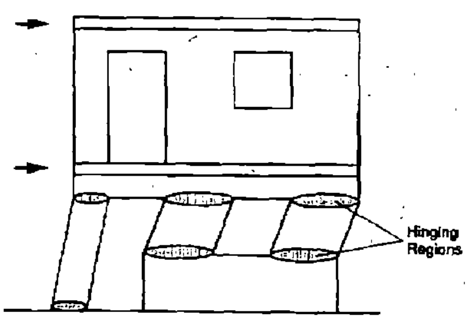
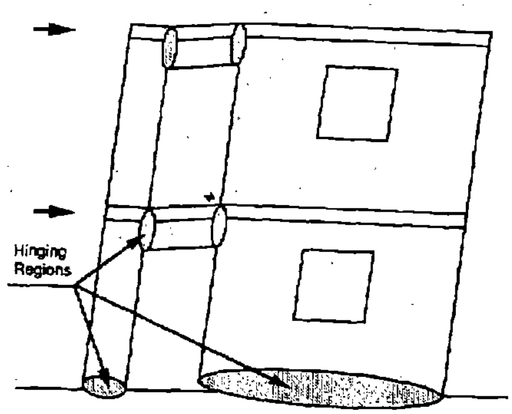
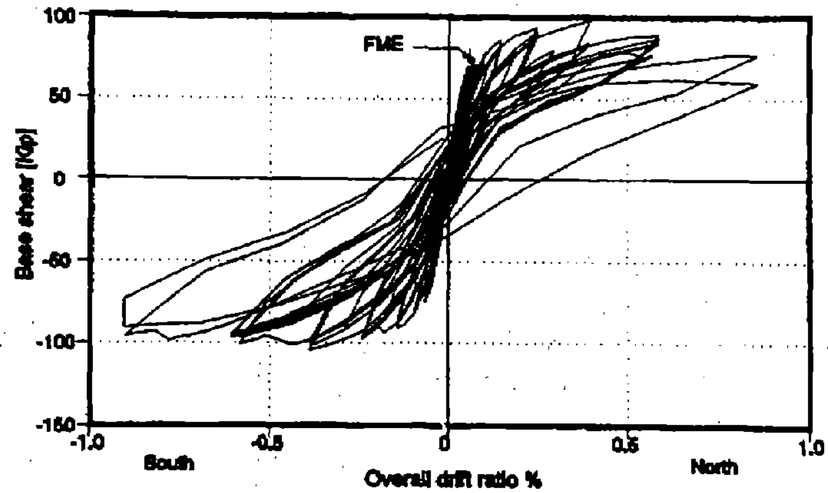


FIGURE 11
COUPLED WALL-TYPE COLLAPSE MECHANISM



For framed structures the coupled-wall mechanism is generally superior, because it leads to a more uniform distribution of inelastic deformation. However, in this case, both types of mechanism showed stable cyclic load-displacement behavior at story drifts approaching 1%. Typical load displacement results are shown in Figure 12.

FIGURE 12
TYPICAL CURVES OF BASE SHEAR VERSUS LATERAL DRIFT FOR PERFORATED WALL SPECIMENS



Based on the test results, the following design approach was recommended for multi-story masonry walls in seismic zones (Leiva 1993, 1994):

1) choose a stable collapse mechanism for the wall, and compute the corresponding lateral load capacity as governed by flexure;

2) provide sufficient flexural reinforcement, distributed in accordance with the collapse mechanism, to resist the design shear;

3) design shear reinforcement according to the "capacity design" approach; and

4) provide suitable details of reinforcement.

ANALYTICAL ADVANCES FROM THE "TCCMAR" PROGRAM

In addition to enhancing our knowledge of experimental behavior of masonry, the TCCMAR program produced advances in analytical modeling of modern masonry structures similar to those studied in that program. The typical analysis has two steps:

1) Using a nonlinear finite-element model, calculate the in-plane, static, load-displacement envelope of the given wall element (Ewing 1990).

2) Idealize the wall as an equivalent, nonlinear, single-degree-of-freedom system. Adjust its hysteretic load-displacement behavior to fit the finite-element envelope determined above, and compute the response of the equivalent system using a lumped-parameter program (Kariotis 1992).

This process is shown in Figures 13 and 14 for a three-story wall. It can be extended to buildings with flexible floor diaphragms and multiple walls.

FIGURE 13
ENVELOPE FROM FINITE-ELEMENT ANALYSIS

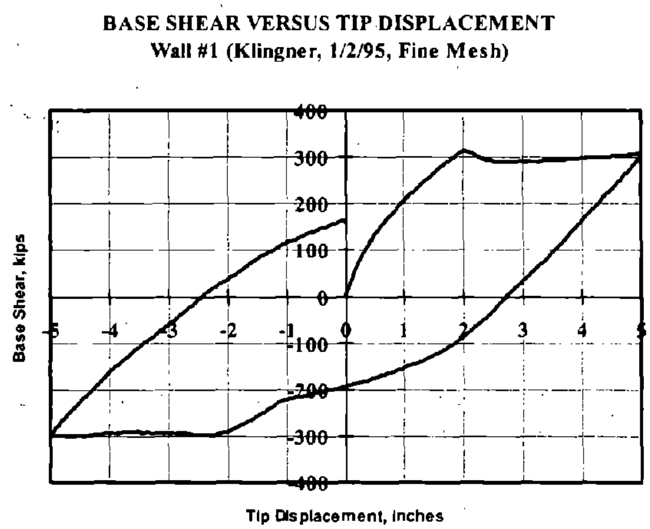
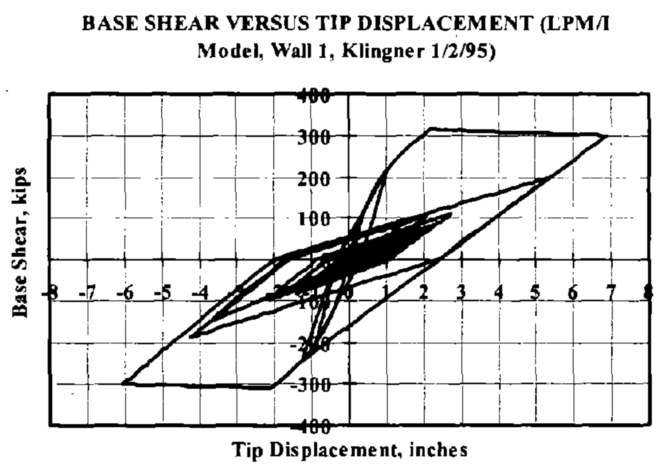


FIGURE 14
LOAD-DISPLACEMENT CURVES FROM LUMPED-PARAMETER MODEL

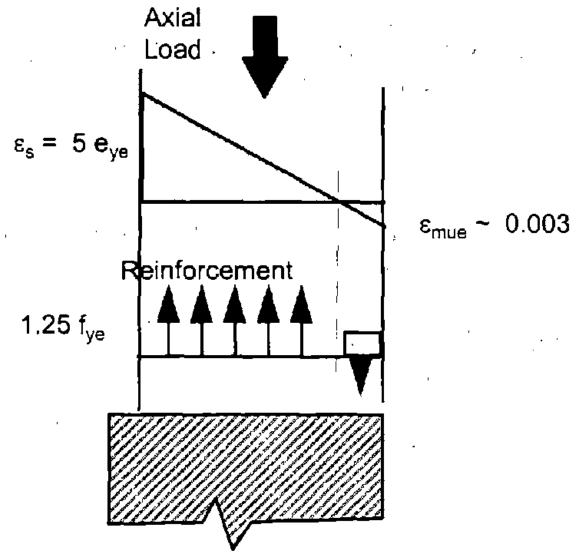


LIMIT-STATE CODE FOR MODERN MASONRY

In the US, current developments in the masonry field include URM retrofitting techniques, a limit-state masonry code, and improvements in material specifications and construction procedures. In the interest of space, only the limit-state masonry code is discussed here.

The limit-state masonry code is now being developed under the auspices of the Masonry Standards Joint Committee (MSJC). It will include the following concepts: yield and capacity limit states; the use of expected values rather than minimum specified ones (to eliminate implicit overstrength); and a strict control on the maximum permissible amount of flexural reinforcement.

FIGURE 15
CRITICAL CONDITION USED TO SET MAXIMUM PERMISSIBLE FLEXURAL REINFORCEMENT



As shown in Figure 10, flexural reinforcement is controlled based on a critical state in which the extreme compression fiber reaches a strain of about 0.003, and the extreme tension reinforcement develops 5 times yield strain. Using plane sections, the neutral axis is located, and corresponding force in the compressive stress block is calculated. The force provided by the tension reinforcement, plus any external compression, must equilibrate that block. This criterion is intended to prevent brittle behavior, and essentially prohibits combinations of axial load and moment above the balance point.

SUMMARY AND CONCLUSIONS

In contrast to the often catastrophic behavior of unretrofitted URM structures, modern reinforced masonry has shown reliable performance against seismic loads. This performance is consistent with research results from the TCCMAR program, and can be predicted using linear and nonlinear analytical models developed by that program. Under the auspices of the Masonry Standards Joint Committee, those results are now being incorporated into draft limit-state design provisions for masonry. Through appropriate control of architectural layout, structural design, material specification, and construction quality, modern masonry is capable of good performance under seismic loads.

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Recent Developments in Nonlinear Modeling of Concrete Structures at Berkeley

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ABSTRACT

The paper presents some recent developments in the nonlinear static and dynamic analysis of concrete structures at Berkeley. After introductory remarks on a new class of element models for the hysteretic behavior of reinforced and prestressed concrete structures, several examples illustrate the present state of research and the capabilities of the proposed models. Future challenges in the nonlinear modeling of concrete structures conclude the paper.

INTRODUCTION

The evaluation of the nonlinear behavior of structures depends on the development of advanced analytical models, which describe the time and load dependent behavior of the structural members. These models should satisfy two basic requirements: (a) they should be reliable, robust and computationally efficient, and, (b) they should be of variable complexity depending on the degree of detail required from the analysis: while individual critical members of the structure need to be evaluated with sophisticated finite element models, the overall behavior of multistory buildings and multiple span freeway structures can be described with sufficient accuracy with simpler member models. In fact, the ability to combine finite element models of critical regions of the structure with nonlinear or even linear member models of the rest of the structure should be an important consideration. Furthermore, the ability to refine a particular element model to the desired degree of detail is another important consideration in the development of such models.

An appropriate platform for the development of structural member models of variable complexity is a general purpose finite element analysis program that meets the following requirements: (a) it allows for an easy and transparent addition of elements to the program, (b) it provides utilities for input data generation, data storage and manipulation and output of the results, but allows the user to also easily incorporate custom-made utilities for new elements, (c) it provides several nonlinear solution strategies for static and dynamic analysis, but allows the user to also include custom-made solution strategies, (d) it provides utilities for, at least, rudimentary graphical pre- and post-processing, and, (e) it is so lean and efficient that it can run on a variety of platforms ranging from personal computers to workstations. Last, but most important, requirement is that the program be capable of accommodating three-dimensional as well as two-dimensional structural models.

The proposed structural element library FEDEAS (Finite Elements for Design, Evaluation and Analysis of Structures) is built around the finite element analysis program FEAP by Robert L. Taylor of the University of California, Berkeley. Salient features of FEAP are documented in Zienkiewicz and Taylor (1989), while a complete manual is in preparation.

This paper presents some salient features of the structural element library FEDEAS. The generality of FEAP makes a distinction between two-dimensional (2D) and three-dimensional (3D) elements unnecessary. Some of FEAP's capabilities are discussed in the examples. The report by Filippou (1997) contains a more comprehensive discussion.

FORMULATION OF STRUCTURAL ELEMENTS

The first release of the structural element library FEDEAS consists of a few elements and a collection of uniaxial hysteretic material models. The proposed elements are three-dimensional, but can be used equally well in a 2D analysis. The elements are: a linear and nonlinear truss element, a linear and nonlinear frame element, and a nonlinear hinge element. In the final development stages is a cable element with nonlinear geometry, a nonlinear frame element for prestressed concrete members with bonded or unbonded tendons, a frame element with relative slip between constituent components that models bond between reinforcing steel and concrete or partial composite action between concrete deck and steel girder.

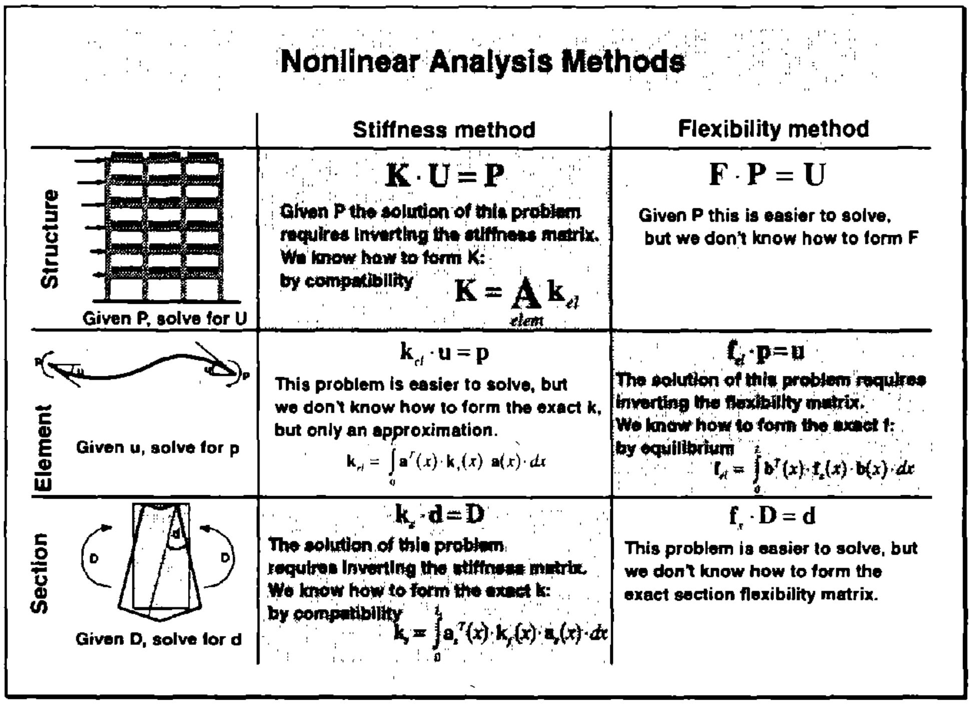


FIGURE 1 - NONLINEAR ANALYSIS METHODS FOR STRUCTURE, ELEMENT AND SECTION

The common characteristic of most of these elements is that they are based on the flexibility method of analysis. In this case the element stiffness matrix is derived by inversion of the flexibility matrix, which is formulated with the virtual force principle: the relation between internal and external work is based on force-interpolation functions that relate the internal forces at a cross section along the element axis to the end forces. This approach offers several advantages over commonly used stiffness-based elements: (a) the force-interpolation functions are exact solutions of equilibrium conditions for the frame element and can, thus, be readily established even in the presence of element loads; this is not the case with stiffness-based models, where the displacement interpolation functions are not exact for nonprismatic and/or nonlinear elements; (b) the strict satisfaction of equilibrium yields superior numerical robustness and accuracy in the presence of strength loss and softening, which can be expected in the evaluation of older concrete structures with poor detailing, but also in new steel structures with fracturing connection behavior; (c) the direct inclusion of element loads yields a significant reduction in the number of nodes and elements of the structural model.

The consistent theoretical framework of the study by Spacone et al. (1996) allows the formulation of a class of flexibility based frame elements with either distributed or concentrated end inelastic deformations. In either case the hysteretic behavior of the section can be described by means of a force-deformation relation or can be derived from the hysteretic behavior of individual fibers into which the section is subdivided (fiber model). In the former case it is really not possible to describe the interaction between internal forces in a rational way, as is the case in the latter at the expense of some complexity.

In the context of distributed inelasticity frame models in a general purpose analysis program that is based on the direct stiffness method of analysis, Figure 1 shows the relation between structure, element and section. It also shows that, while the stiffness or flexibility method might provide the straightforward solution of the nonlinear relation between forces and displacements depending on which level of structural discretization the method is used for, the rational formulation of the problem based on statements of compatibility or equilibrium, invariably requires the inversion of the corresponding matrix. This rational path is highlighted in Figure 1, showing clearly the importance of the flexibility method in element formulation, for which the equilibrium conditions readily furnish the force interpolation functions. By contrast, the stiffness method is clearly superior at the structure as well as at the section level, as amply demonstrated by its popularity in structural analysis programs. The concept of using force interpolation function in the element state determination underlies the formulation of most structural elements in FEDEAS.

MATERIAL LIBRARY

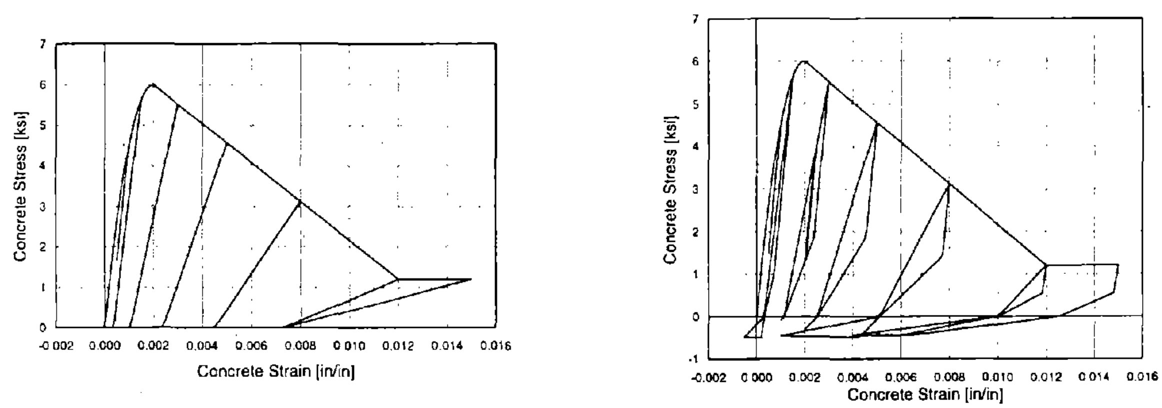


FIGURE 2-HYSTERETIC STRESS-STRAIN RELATION OF CONCRETE MATERIAL MODELS

The material library of FEDEAS consists of uniaxial force-deformation relations that describe the hysteretic behavior of fibers or sections of the structural elements. Several models of the same material are available, allowing the user to select the desired level of complexity. There are, thus, three models for the hysteretic behavior of concrete: (a) a model with no tensile strength, (b) a model with tensile strength and a linear tensile strain softening branch, and (c) a model with tensile strength and a nonlinear tensile strain softening branch. All three models have the same behavior in compression. Model (a) has a simple rule for loading-unloading in compression, while models (b) and (c) follow a slightly more complex rule. Figure 2 shows a typical stress-strain history for concrete models (a) and (b).

The material library also contains several hysteretic steel models. Figure 3 shows the characteristic hysteretic behavior for two of these: a bilinear model with isotropic strain hardening in tension and compression and a nonlinear steel model according to Giuffré-Menegotto-Pinto modified to include the same isotropic strain hardening as the bilinear model.

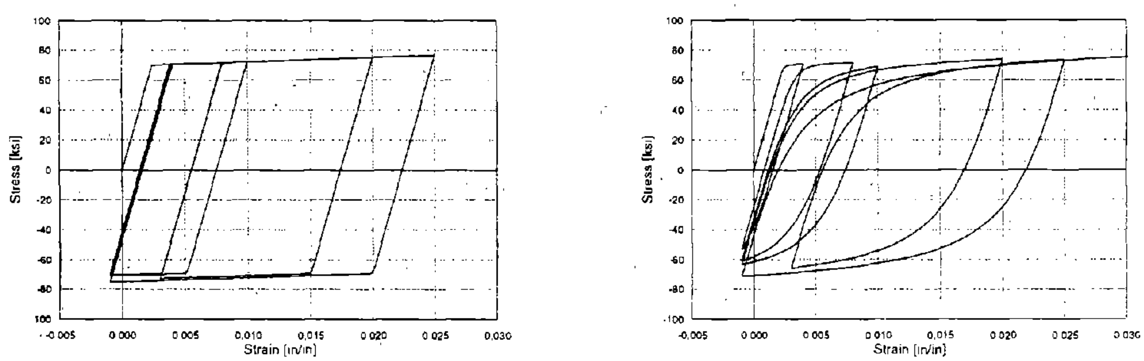


FIGURE 3-HYSTERETIC STRESS-STRAIN RELATION OF STEEL MATERIAL MODELS

Finally, the library includes several generic hysteretic force-deformation models that can be either used in the modeling of individual fibers or in the modeling of the force-deformation behavior of plastic hinges and the section force-deformation behavior of distributed inelasticity frame elements. Two examples, one from a model with bilinear envelope and one from a model with trilinear envelope are shown in Figure 4. The model with the trilinear envelope is shown with a negative (softening) second slope in the negative force-deformation quadrant.

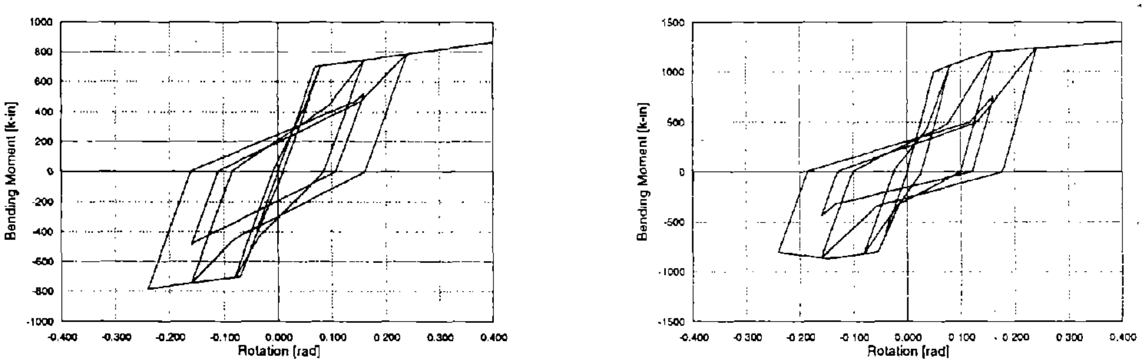
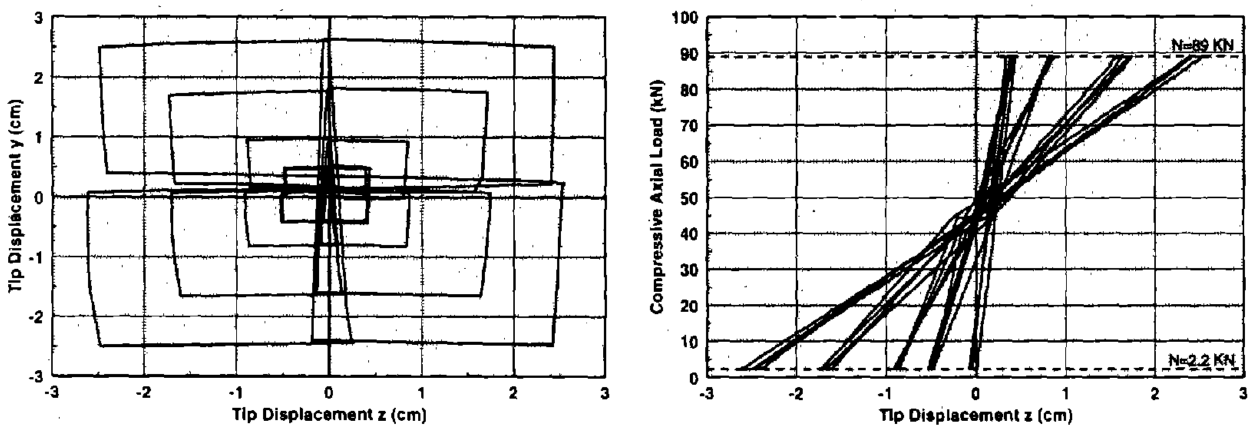


FIGURE 4- GENERIC HYSTERETIC MODELS FOR FIBERS OR CROSS SECTIONS

EXAMPLES

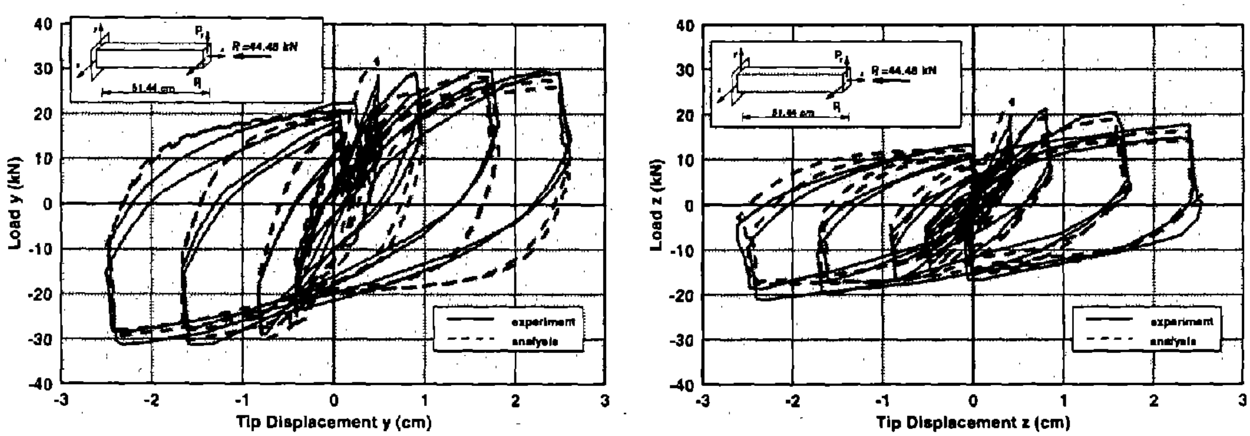
REINFORCED CONCRETE COLUMN

The first example concerns the hysteretic behavior of a rectangular reinforced concrete cantilever. Specimen #5 in the test series of Low and Moehle (1987) was subjected to a rather complex load history of biaxial bending with variable axial force intended to simulate loading conditions in structures under bi-directional acceleration input and torsional effects. The imposed tip displacement history is shown in Figure 5a and the variation of axial force in Figure 5b. The analytical model consists of a fiber beam-column element with three control sections. The subdivision of the section in concrete and steel fibers is advisable here on account of the variability of the axial force and the imposed biaxial displacement history.



(a) (b)

FIGURE 5 LOAD HISTORY FOR MODEL OF LOW-MOEHLE SPECIMEN #5: (A) IMPOSED TIP DISPLACEMENTS; (B) APPLIED AXIAL LOAD



(a) (b)

FIGURE 6 COMPARISON OF ANALYTICAL WITH EXPERIMENTAL RESULTS FOR TIP LOAD-DISPLACEMENT RELATION: (A) Y-DIRECTION; (B) Z-DIRECTION

The model was subjected to the measured lateral displacement history and the corresponding axial force at the column tip. The agreement between experimental and analytical force-displacement relation in Figure 6 is very satisfactory, even though the discrepancy in the "pinching" behavior of the load displacement relation is noticeable. A comparison of the measured strains at two corner reinforcing bars with calculated fiber strains in Figure 7 shows excellent qualitative agreement, but the maximum tensile strains are off by a factor of 2 or more (note the different strain scale in the side-by-side figures). It can, thus, be concluded that the effects of shear and bond-slip (pull-out) of longitudinal reinforcing steel play a significant role in the local response of reinforcing steel strains. Thus, an analytical model needs to address these effects before attaining "predictive" abilities for the failure mode of the specimen, which depends on steel and concrete strains (buckling, crushing, spalling).

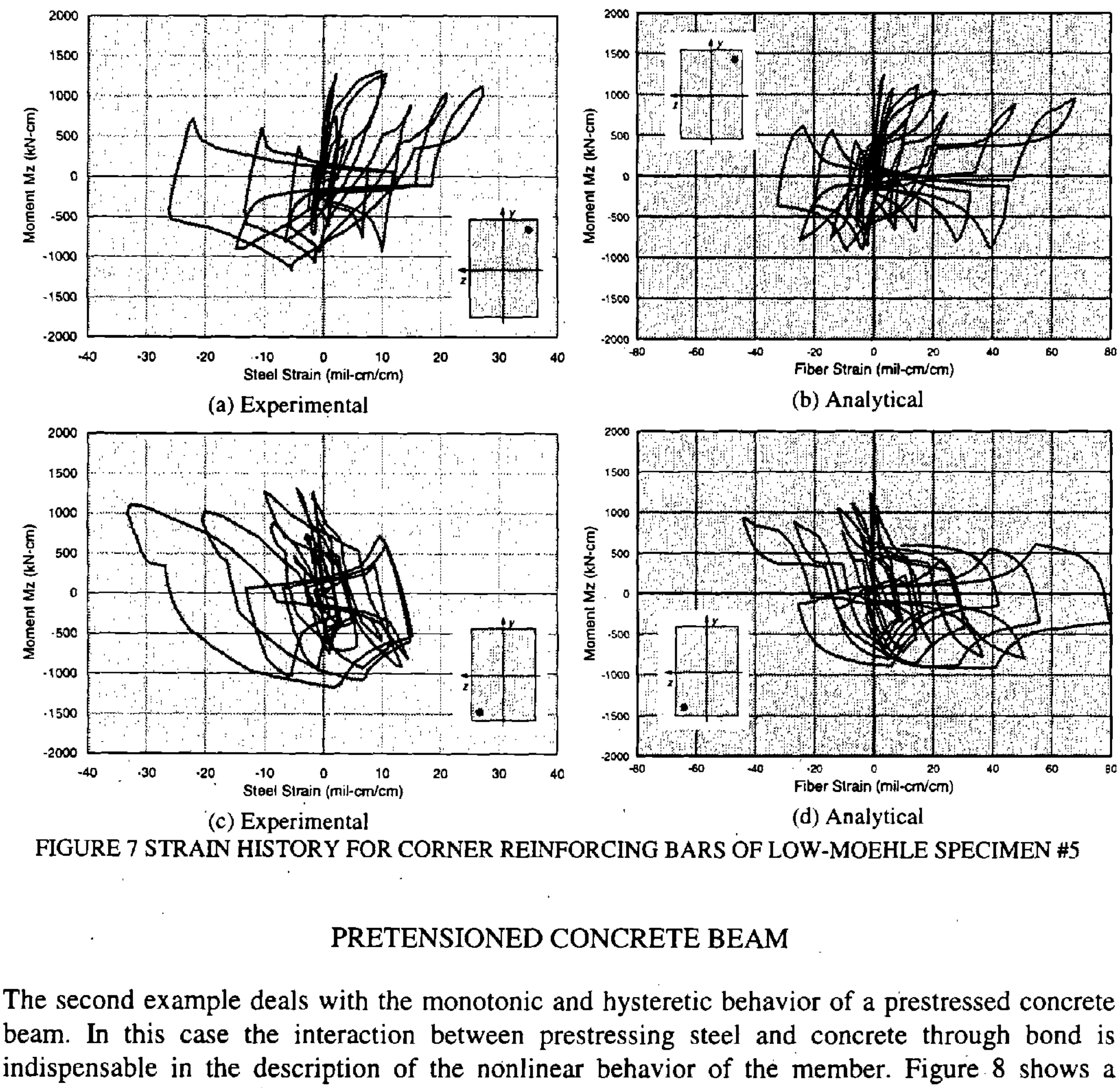


FIGURE 7 STRAIN HISTORY FOR CORNER REINFORCING BARS OF LOW-MOEHLE SPECIMEN #5

PRETENSIONED CONCRETE BEAM

The second example deals with the monotonic and hysteretic behavior of a prestressed concrete beam. In this case the interaction between prestressing steel and concrete through bond is indispensable in the description of the nonlinear behavior of the member. Figure 8 shows a beam-column element with bond elements that connect the tendon with a fiber beam-column element that simulates the behavior of concrete and bonded reinforcing steel. For unbonded tendons the bond elements are located at the physical ends of the prestressing tendon and are endowed with special characteristics for simulating the construction sequence of jacking (no

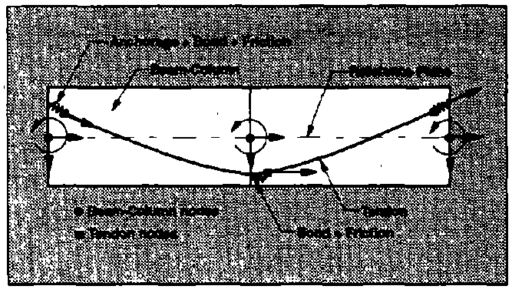


FIGURE 8 PRESTRESSED CONCRETE ELEMENT WITH BOND

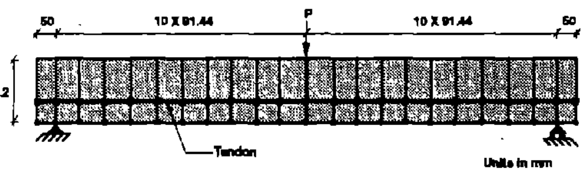


FIGURE 9 MODEL OF PARANAGAMA BEAM

bond stiffness) and release/anchorage (bond-slip stiffness according to anchorage device). The anchorage subelement is thus a very stiff linear spring acting in the tangential direction of the tendon and the axial deformation of the spring represents the slip of the tendon end relative to the beam-column. Intermediate smeared bond elements can be arranged along the prestressing tendon to simulate grouting, friction losses and bond-slip under loading (Mohd Yassin 1994).

Fig. 9 shows the finite element model of a pretensioned bonded prestressed concrete beam tested by Paranagama et al (1969) to investigate the effect of a small number of repeated loadings on the moment-deformation characteristics. Smeared bond elements are used to model the transfer bond at release and the flexural bond during loading. The experimental and analytical midspan moment-curvature response, as shown in Fig. 10, exhibit good agreement for the response envelope. Although no reloading curve is shown for the experiment, it is known that reloading shows a slightly stiffer response than unloading. On the other hand, the analysis produces significantly better recovery both in terms of residual deformation and reloading strength. This behavior is expected because the model does not include

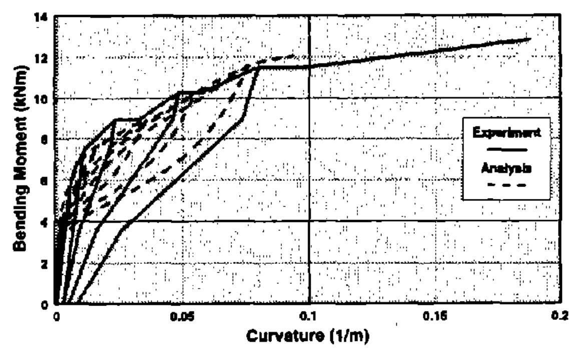


FIGURE 10 MIDSPAN MOMENT-CURVATURE FOR PARANAGAMA BEAM

the accumulation of bond and concrete damage under repeated loading cycles. However the basic shape of the unloading curve is approximated quite well by the model. Fig. 11 shows the analytical tendon strain distribution at various stages of loading. The pattern of behavior correlates very well with experimental results and proves the usefulness of the model as a tool to study development lengths and bond.

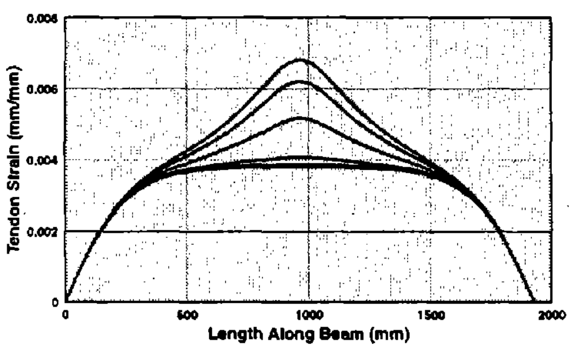


FIGURE 11 TENDON STRAIN DISTRIBUTION FOR PARANAGAMA BEAM

SUMMARY AND CONCLUSIONS

Recent developments in the nonlinear modeling of concrete structures have improved our ability to simulate the their response under extreme loading conditions. In spite of this progress, much remains to be done before we might be able to simulate the local response and failure mode of elements and structures under cyclic loading conditions. In this respect, material modeling deserves attention, especially, after concrete cracking. More importantly, the interplay between flexure, axial force and shear and the bond-slip of reinforcing steel are difficult to account for in a rational, simple manner in a beam-column element suitable for the analysis of large structures. In bridge elements the effect of torsion complicates matters further. The need to combine simplicity with accuracy in the development of improved structural elements for the evaluation of existing and the analysis of new structures is bound to challenge researchers in the years to come.

ACKNOWLEDGEMENTS

The author wishes to acknowledge the teaching and research guidance of Professor V.V. Bertero during his graduate studies at the University of California. Fruitful and enjoyable discussions in the intervening years have always helped to clarify his ideas and steer his efforts to a better modeling of the hysteretic behavior of concrete structures.

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Damage-Controlled Earthquake Resistant Design Method Based on the Energy Concept

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SUMMARY

Previous seismic design methods have been developed with structural safety as the major consideration, while performance in other areas has been neglected, with the following results.

- During the severe earthquakes, structure are inevitably damaged to some extent and sometimes, repair is very expensive.
- Strengthening structures in order to reduce structural damage results in an increased acceleration response, which causes overturning of furniture and equipment. Thus, it causes an interruption of daily activities such as medical treatment and results in the loss of property.

On the other hand, the recently developed base-isolation technique has overcome the above-mentioned difficulties, without deterioration of the performance obtained using conventional earthquake-resistant design methods.

In this paper, an earthquake resistant design method for buildings which meet the requirements for both structural safety and reduction of the acceleration response is discussed.

The proposed design method is consistent to the method applied to base-isolated structures and is developed based on the balance between the seismic energy input and the energy absorption capacity of the structure. Structures in general are very complicated and prediction for exact behavior of them is, sometimes, very difficult. Therefore, in order to develop the performance-based design method, it is also necessary to exploit preferable structural types of which prediction of structural behavior can be explicitly made.

As a preferable structural type, the flexible-stiff mixed structure is introduced. The flexible-stiff mixed structure consists of the flexible elements which remain elastic even under severe earthquakes with a relatively low elastic rigidity and the stiff elements which behave mainly plastically with a relatively high elastic rigidity.

Conventional type of multi-story buildings can be modeled to be a flexible-stiff mixed structure by definitely allotting a role of the flexible element or the stiff element to each structural element. Major damage indices such as the cumulative plastic deformation, the maximum deformation, the residual deformation and the maximum yield shear force coefficient are clearly related to the level of seismic input.

1. FLEXIBLE-STIFF MIXED STRUCTURES

The structure which is composed of the flexible elements remained elastic and the stiff elements with a high elastic rigidity and a high plastic deformation capacity is defined as the flexible-stiff mixed structure (Akiyama, 1985), (Akiyama, 1995). In flexible-stiff mixed structures, the yield strengths in positive and negative loading domains, $|Q_Y^+|$ and $|Q_Y^-|$, become different as the deformation develops. Generally, cumulative plastic deformations are liable to concentrate in the element with a relatively weak yield strength. Therefore, a further development of the plastic deformation in a loading domain where plastic deformations have developed with an increase of the yield strength is restrained antonomously in the flexible-stiff mixed structure, thus resulting in equalization of deformations in positive and negative loading domains. Main features in the response characteristics of the flexible-stiff mixed structures are summarized as follows.

1) The cumulative plastic defamations in positive and negative loading domains are nearly equal.
2) The maximum deformation in positive and negative loading domains are nearly equal.
3) Efficiency of the energy adsorption with respect to a maximum deformation is high.
4) The residual deformation can be made considerably small.

Referring to these characteristics, in comparison to the ordinary structures consisting of monotonous elastic-plastic elements, the flexible-stiff mixed structures are considered to be a preferable structural type of which performance in the seismic resistance can be clearly stated.

The cumulative plastic deformation, δ_p, is related to the maximum deformation, δ_m, by the following empirical equation in the flexible-stiff mixed structure, neglecting the elastic deformation of the rigid element:

$$\delta_p = 8\,\delta_m \tag{1}$$

Also, the residual deformation in the flexible-stiff mixed structure, δ_r, is expressed empirically as:

$$\delta_r = 0.2\,{}_sQ_y\left(\frac{1}{{}_fk} - \frac{1}{{}_sk}\right) \quad , \qquad \text{and also} \qquad \delta_r < \delta_m \tag{2}$$

where ${}_sQ_y$: the yield strength of the rigid element
${}_fk$: the rigidity of the flexible element
${}_sk$: the rigidity of the stiff element

2. RESPONSE OF THE FLEXIBLE-STIFF MIXED STRUCTURE

The total energy input in a structure exerted by an earthquake is a very stable amount and scarcely influenced by distributions of the mass, the stiffness and strength, and depends mainly on the total mass and the fundamental natural period of the structure. The total energy input can be converted to the equivalent velocity, V_E, by the following equation:

$$E = \frac{M\,V_E^2(T)}{2} \tag{3}$$

where E : the total energy input
T : the fundamental natural period
M : the total mass of structure

The V_E - T relationship is defined as the energy spectrum. The energy spectrum for the single-mass system with the damping of 10% of the critical damping i,e., $h = 0.1$, can be considered to be the energy spectrum for design use (Akiyama, 1985).
The form of the energy spectrum can be represented by a bilinear curve shown in **Figure 1**. That is, the V_E - T relationship is expressed by a line passing through the point of origin in the short-period range and takes a constant value in the long-period range as is expressed by:

$$\begin{aligned} &\text{for } T \le T_G, \quad V_E = \frac{V_{Em}}{T_G} T \\ &\text{for } T > T_G, \quad V_E = V_{Em} \end{aligned} \tag{4}$$

where V_{Em} : the maximum value of V_E

The energy input attributable to the structural damage, E_D, is also converted to the equivalent velocity, V_D, through the equation similar to Eq. (4).
V_D is related to V_E by the following empirical formula:

$$V_D = \frac{V_E}{1 + 3h + 1.2\sqrt{h}} \tag{5}$$

The flexible-stiff mixed structure is assumed to be a shear type of multistory frame. The restoring force characteristics of the stiff element in each story is assumed to be elastic-perfectly plastic type. A hysteretic behavior of one story is shown in **Figure 2**. The rigidity of the stiff element is denoted by ${}_sk$ and the rigidity of the flexible element is denoted by ${}_fk$. ${}_s\delta_y$ is the yield deformation of the stiff element. Under the maximum deformation, δ_m, the instantaneous period of vibration of the system takes a value of T_m. The secant rigidity, k_e, associated with δ_m can be applied in order to predict T_m by using the following formula:

$$T_m = 2\pi\sqrt{\frac{M}{k_e}} \tag{6}$$

The energy attributable to the damage can be expressed in terms of potential energy under the gravity field i.e., by the equivalent height of the mass, h_E, according to the following equation:

$$h_E = \frac{E_D}{Mg} = \frac{V_D^2}{2g} \tag{7}$$

where g : the acceleration of gravity
Generally, the energy input attributable to the damage is finally absorbed by structural skeletons of a structure in a form of cumulative plastic deformation. Therefore, the following equation holds:

$$E_D = \sum_{i=1}^{N} W_{pi} \tag{8}$$

where W_{pi} : the cumulative plastic strain energy in each story
N : the number of the story

Eq.(8) can be written in respect to the damage of the first story as:

$$E_D = W_{p1} \gamma_1 \tag{9}$$

where γ_1 : the ratio of the total damage to W_{p1}

γ_1 can be expressed as (Akiyama, 1985):

$$\gamma_1 = 1 + p_o^{-n} \sum_{i=2}^{N} s_i \quad , \qquad p_o = 1.185 - 0.0014N \tag{10}$$

where p_o : the deviation of the actual strength distribution from the strength distribution aimed at in the design
s_i : the value which is determined by distributions of the mass, the rigidity and the strength of the structure
n : the power which reflects the damage concentration characteristic of the structure

It is assumed that the mass of each story is constant and the yield deformation of the rigid element of each story, ${}_s\delta_y$, is constant. To such a system, the following empirical formula is applied:

$$\sum_{i} s_i = 0.36 + 0.64\, N \tag{11}$$

Therefor, Eq.(10) is reduced to:

$$\gamma_1 = 1 + 0.64\,(N - 1)\, p_o^{-n} \tag{12}$$

By dividing Eq. (9)with M g , the following equation is obtained:

$$h_E = {}_s\alpha_{Y1}\, \delta_{p1}\, \gamma_1 \tag{13}$$

where ${}_s\alpha_{Y1} = {}_sQ_{Y1} / M g$: the yield shear force coefficient in the first story

Using Eqs.(1) and (13), ${}_s\alpha_{Y1}$ is determined as:

$${}_s\alpha_{Y1} = \frac{h_E}{8\, \gamma_1\, \delta_{m1}} \tag{14}$$

where δ_{m1} : the maximum displacement of the first story

The fundamental natural period of the shear-type system can be written in terms of the spring constant of the first story, k_1, as:

$$T = 2\pi\sqrt{\frac{M\,\kappa_1}{k_1}} \tag{15}$$

where $\kappa_1 = k_1 \;/\; k_{eq}$

k_{eq} : the equivalent spring constant of the single-degree of freedom system with M and T

κ_1 for the system herein dealt with can be approximated by:

$$\kappa_1 = 0.48 \;+ 0.52\;N \tag{16}$$

While in the long-period range, the energy input is given regardless of the value of the period, the energy input in the short-period range depends on the period. Therefore, the period must be precisely estimated,

The substantial period for the system of which the period of vibration changes between T_o and T_m is calculated for the short-period range as:

$$T_e = \sqrt{\frac{T_o^2 + T_o\,T_m + T_m^2}{3}} \tag{17}$$

where T_e : the substantial period of the system

T_o : the period in the elastic range

Referring to **Figure 2**, T_o and T_m are written as:

$$T_o = 2\pi\sqrt{\frac{\kappa_1}{g\left(\frac{{}_s\alpha_1}{{}_s\delta_{Y1}} + \frac{{}_f\alpha_1}{\delta_{m1}}\right)}} \quad , \quad T_m = 2\pi\sqrt{\frac{\kappa_1}{g\left(\frac{{}_s\alpha_1 + {}_f\alpha_1}{\delta_{m1}}\right)}} \tag{18}$$

where ${}_f\alpha_1 = \frac{{}_fk_1\,\delta_{m1}}{M\,g}$: the shear force coefficient of the flexible element in the first story

3. ILLUSTRATIVE EXAMPLE

As an illustrative example, the system in which ${}_sk$ is sufficiently greater than ${}_fk$ and ${}_s\delta_Y$ is negligible small, is taken.

Applying these assumptions, T_o becomes nullified, and T_e is reduced to:

$$T_e = \frac{2\pi}{\sqrt{3}}\sqrt{\frac{\kappa_1\,\delta_{m1}}{(1+f)\;{}_s\alpha_{Y1}}} \tag{19}$$

where $f = {}_f\alpha_1 \;/\; {}_s\alpha_{Y1}$

Denoting h_E at $T = T_G$ by h_{Em}, h_E in the short-period range characterized by the linear V_D - T relationship is written as:

$$h_E = h_{Em}\left(\frac{T_e}{T_G}\right)^2 \tag{20}$$

Using Eqs.(14) (19) and (20), ${}_s\alpha_{Y1}$ for the energy input in the short-period range is determined as follows, regardless of δ_{m1}:

$${}_s\alpha_{Y1} = \frac{\pi}{\sqrt{6}\,T_G}\sqrt{\frac{h_{Em}\,\kappa_1}{(1+f)\,g\,\gamma_1}} \tag{21}$$

On the other hand. in the long-period range, since $h_E = h_{Em}$, ${}_s\alpha_{Y1}$ is obtained as:

$$\alpha_{Y1} = \frac{h_{Em}}{8\,\gamma_1\,\delta_{m1}} \tag{22}$$

As an illustrative example, the maximum level of the ground motion which competes with the Hyogoken-nanbu earthquake, 1995 is applied, i.e., T_G and V_{Em} in the energy spectrum shown in **Figure 1** are selected to be:

$$\begin{aligned} V_{Em} &= 400\text{cm} \\ T_G &= 1.0\text{sec} \end{aligned} \tag{23}$$

The damping of $h = 0.02$ is assumed. Then, the maximum value of V_D, V_{Dn}, and h_E become as:

$$\begin{aligned} V_{Dm} &= \frac{V_{Em}}{1 + 3\,h + 1.2\sqrt{h}} = 325\ \text{cm/sec} \\ h_{Em} &= 54\ \text{cm} \end{aligned} \tag{24}$$

As a structural performance, the maximum story displacement in the first story is assumed to be:

$$\delta_{m1} = 5\text{cm},\quad 6.67\text{cm},\quad 10\text{cm} \tag{25}$$

A weak-beam type of structure is assumed, that is, in estimating the damage distribution, n = 6.0 is taken.

${}_s\alpha_{Y1}$ obtained by Eq.(21) is denoted by $({}_s\alpha_{Y1})_I$ and ${}_s\alpha_{Y1}$ obtained by Eq.(22) is denoted by $({}_s\alpha_Y)_{II}$. The smaller of those becomes the real value of ${}_s\alpha_{Y1}$ which corresponds to the given energy spectrum. The larger value of those corresponds to the extended lines of the two line segments of the energy spectrum. The ${}_s\alpha_{Y1}$ - N relationships for f = 1.0 are shown in **Figure 3**, in which the solid lines are valid due to the above-mentioned reason.

Also, in **Table 1**, the values of ${}_s\alpha_{Y1}$ for f = 1.0 are shown. The values of $({}_s\alpha_{Y1})_{II}$ listed above the horizontal line are larger than $({}_s\alpha_{Y1})_I$. Accordingly, those value are not valid. In the short-period range, ${}_s\alpha_{Y1}$ takes a constant value irrespective of δ_{m1}. In this case, however , δ_{m1} is limited by the condition that T_e does not exceed T_G. Denoting δ_{m1} which corresponds to T_G by $\bar{\delta}_{m1}$, $\bar{\delta}_{m1}$ is written as:

$$\bar{\delta}_{m1} = \frac{3\, g_s\, \bar{\alpha}_{Y1}\, T_G^2}{(1+f)\,\pi^2\,\kappa_1} = \frac{\sqrt{3}\, T_G}{2(1+f)\,\pi} \sqrt{\frac{g\, h_{Em}}{\kappa_1\, \gamma_1}} \tag{26}$$

In the short period range, an arbitrary value of δ_{m1} can be taken under a constant value of ${}_s\alpha_{Y1}$. However, under the given condition ${}_f\alpha_{Y1} = f\, {}_s\alpha_{Y1}$, ${}_fk_1$ must be:

$$ {}_fk_1 = \frac{f\, {}_s\alpha_{Y1}}{\delta_{m1}} \tag{27}$$

Actually, the restraing condition of Eq.(27) can be mitigated, that is, the rigidity higher than that given by Eq(27) can be allowed on the reason that a higher rigidity makes T_e smaller than the prescribed value due to Eq.(27), resulting in a decrease of the energy input in the short-period range. In the region of $\delta_{m1} > \bar{\delta}_{m1}$, ${}_s\alpha_{Y1}$ is given by $({}_s\alpha_{Y1})_{II}$ and the rigidity is secured also by Eq.(27). Assuming that ${}_fk\,/\,{}_sk$ is negligibly small and substituting ${}_sQ_Y = {}_fQ_Y / f$ into Eq.(2), the residual plastic deformation in the first story is obtained as:

$$\delta_{r1} < \frac{0.2\,\delta_{m1}}{f} \text{ , and also } \delta_{r1} < \delta_{m1} \tag{28}$$

4. CONCLUSION

Performance of structures subjected to earthquakes can be stated in terms of the energy input spectrum, the cumulative plastic deformation, the maximum deformation, the residual deformation and the maximum acceleration response. The maximum acceleration response can be estimated based on the yield shear force coefficient. The most advanced type of structures with respect to structural performance is the base-isolated structure. The flexible-stiff mixed structure is a generalized structural type of the base-isolated structure and can be realized in conventional multi-story buildings.

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TABLE 1
THE $_s\alpha_{Y1}$ - N RELATIONSHIP

N	γ_I	κ_I	$(_s\alpha_{yI})_I$	$\delta_{mI}(cm)$	$(_s\alpha_{yI})_I$ $\delta_{mI}(cm)$ 10.0	6.67	5.0
1	1.00	1.00	0.213	31.7	0.675	1.011	1.350
2	1.23	1.52	0.237	23.2	0.548	0.822	1.096
3	1.47	2.04	0.251	18.3	0.459	0.688	0.918
4	1.71	2.56	0.261	15.1	0.394	0.591	0.788
5	1.96	3.08	0.267	12.9	0.344	0.516	0.688
6	2.21	3.60	0.272	11.2	0.305	0.458	0.610
8	2.71	4.64	0.279	8.9	0.249	0.373	0.498
10	3.23	5.68	0.285	7.4	0.209	0.313	0.418
12	3.77	6.72	0.284	6.3	0.179	0.268	0.358
14	4.32	7.76	0.289	5.5	0.156	0.234	0.312
16	4.89	8.80	0.286	4.8	0.138	0.207	0.276
18	5.47	9.84	0.286	4.3	0.123	0.185	0.247
20	6.07	10.88	0.285	3.9	0.111	0.167	0.222
22	6.68	11.92	0.283	3.6	0.101	0.151	0.202
24	7.32	12.96	0.283	3.25	0.092	0.138	0.184
26	7.97	14.00	0.282	3.0	0.085	0.127	0.169
28	8.64	15.04	0.281	2.8	0.078	0.117	0.156
30	9.32	16.08	0.280	2.6	0.072	0.109	0.145

FIGURE 1
ENERGY SPECTRUM

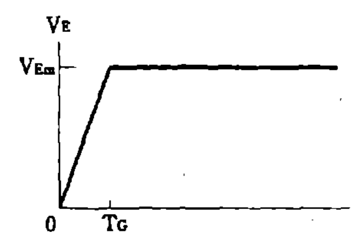


FIGURE 2
HYSTERESIS LOOP

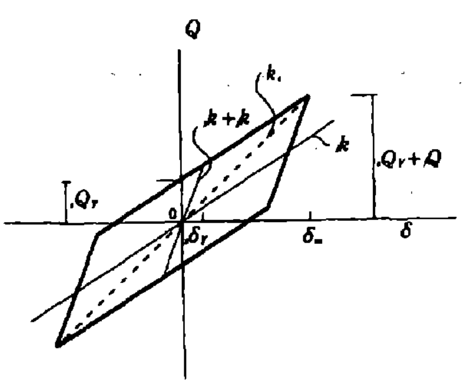
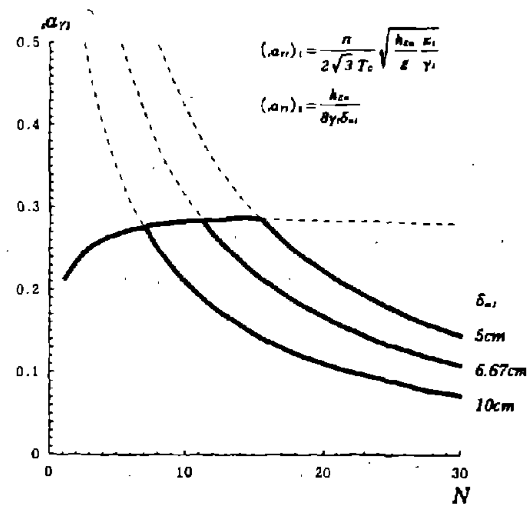


FIGURE 3
THE $_s\alpha_{Y1}$ - N RELATIONSHIP



Lessons from Steel Buildings Damaged by the Northridge Earthquake

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SUMMARY

One of the important overall surprises of the Northridge earthquake of January 17, 1994, was the widespread and unanticipated brittle fractures in welded steel beam to column connections. The economy, versatility and presupposed high plastic deformation capacity of welded steel moment-resisting frame (WSMF) buildings led to their common usage in Los Angeles and elsewhere in the U.S. No casualties or complete collapses occurred during the Northridge earthquake as a result of these connection failures, and WSMF buildings in areas of moderate shaking were not damaged at all. However, a wide spectrum of brittle connection damage did occur, ranging from minor cracking observable only by nondestructive testing to completely severed columns.

This paper reviews the performance of steel buildings during the Northridge earthquake and the implications for design practice. Some of the results of studies undertaken as part of a project initiated by U.S. Federal Emergency Management Agency (FEMA) to reduce the earthquake hazards posed by steel moment-resisting frame buildings. The objective of this project is to develop and verify reliable and cost-effective methods for the inspection, evaluation, repair, and rehabilitation of existing steel frame buildings and for the construction of new ones.

INTRODUCTION

Every earthquake provides new lessons for the earthquake engineering profession. The widespread damage to welded steel moment resisting frame systems was one of the major overall lessons of the Northridge earthquake. The brittle nature of the fractures detected in numerous welded steel beam to column connections, essentially invalidated historic design approaches and code provisions based on "ductile" structural response.

The most commonly observed damage occurred in or near the welded joint of a girder bottom flange to the supporting column flange; complete brittle fractures occurred in many cases. Damage was so severe in some buildings that all of the moment resisting connections at one or more floors failed, or significant permanent lateral displacements occurred. In one case, damage was so severe the building was demolished, and several buildings were evacuated.

Thus far, more than 150 damaged buildings have been identified, including hospitals and other health care facilities, government, civic and private offices, cultural and educational facilities, residential structures, and commercial and industrial buildings. Damage occurred in new as well as old buildings; in tall as well as in short structures. While inadequate workmanship was believed to play a major role in the damage observed in some structures, most damaged buildings are believed to be constructed consistent with modern codes and standards of practice. The effect of these observations has been a loss of confidence in the procedures used in the past to design and construct welded connections in steel moment frames, and a concern that existing structures incorporating these connections may not be sufficiently safe.

A particularly disconcerting aspect of this damage is that it often occurred without accompanying distress to architectural finishes and cladding. As a result, reconnaissance reports immediately following the Northridge earthquake often cited the apparent excellent behavior of steel frame buildings. However, severe damage found in buildings under construction at the time of the earthquake, and detailed investigations of WSMF buildings which suffered increasing amounts of damage during aftershocks, quickly identified the true performance.

Current professional judgment is that the historic practices used for the design and construction of WSMF connections do not provide adequate reliability and safety, and should not continue to be used in the construction of new buildings intended to resist earthquake ground shaking through inelastic behavior. As a consequence, pre-qualified connection details and design methods contained in the major U.S. building codes have been rescinded, and emergency code provisions stipulate that new designs be substantiated by testing or test-verified calculations. Several fundamental questions must be answered in order to develop effective and economical design procedures and construction standards, and to restore public and professional confidence in this form of construction. These questions include:

- What happened to WSMF buildings during the Northridge earthquake?
- What caused the observed damages?
- How to identify WSMF buildings that may have sustained damage?
- How safe are damaged WSMF buildings and do they need to be repaired?
- How can damaged buildings be reliably repaired and/or upgraded?
- How to design and construct new buildings so they will not sustain similar damage?
- Can the vulnerability of existing WSMF buildings to future earthquakes be reliably determined and mitigated through effective rehabilitation procedures?
- What are the economic, social and political costs of new design or construction practices?

Answering these questions involves consideration of many complex technical, professional and economic issues including metallurgy, welding, fracture mechanics, connection behavior, system performance, and practices related to design, fabrication, erection and inspection. Unfortunately, current knowledge is inadequate.

PROGRAM TO REDUCE EARTHQUAKE HAZARDS IN STEEL MOMENT FRAME STRUCTURES

A coordinated, problem-focused program of research, investigation and professional development has begun under FEMA-sponsorship develop reliable, practical and cost-effective guidelines and standards of practice related to steel moment-resisting frame buildings for:

1. the identification, inspection and rehabilitation of existing at-risk buildings prior to a damaging earthquake,
2. the identification, inspection, and repair or upgrading of damaged buildings following an earthquake, and
3. the design and construction of new buildings.

This program is being managed by the SAC Joint Venture comprised of the Structural Engineers Association of California, Applied Technology Council and California Universities for Research in Earthquake Engineering. However, all aspects of the program are conducted with active involvement of design and construction experts, researchers and others from throughout the U.S.

The Steel Program is divided into two major phases. The first phase focused on the development of Interim Guidelines [1] for the inspection, evaluation, repair, modification and construction of welded steel structures. This phase was supported by limited amounts of laboratory and field testing, as well as focused investigations. Major efforts to identify and verify reliable and cost-effective long term solutions and to develop seismic design criteria for steel frame structures are contained in the second phase.

The backbone of the Steel Program is the development of specific design advisories, guidelines and other criteria for design, inspection, evaluation, repair, modification and rehabilitation of steel moment frame structures. The Interim Guidelines [1] developed in Phase 1 were written by ten experts from a variety of disciplines. The Guidelines were subjected to extensive review by engineers, researchers, building regulators and other public officials, and representatives from the steel and construction industries. The scope of the Interim Guidelines covers welding procedures, quality assurance, post-earthquake actions, and new construction. Specific chapters cover: (a) welding and metallurgy; (b) quality control and assurance; (c) visual inspection; (d) non-destructive testing; (e) classification and implications of damage; (f) post-earthquake evaluation; (g) post-earthquake inspection; (h) post-earthquake repair and modification; and (i) new construction.

Some of the results of the preliminary investigations and tests carried out to support the development of the Interim Guidelines and the planning of Phase 2 are described below. Additional information on these investigations can be found in References 2 through 14. Information on activities being undertaken in Phase 2 is presented at the end of the paper.

SURVEY OF NORTHRIDGE STEEL BUILDING DAMAGE

Four types of surveys were used to assess the damage to steel frame buildings caused by the Northridge earthquake. In the first, in-depth interviews [2] were conducted with design engineers, building inspectors, contractors and building officials. A number of important difficulties were detected in identifying damaged buildings and in inspecting and repairing them.

In the second type of survey, a brief questionnaire was sent to more than two hundred, randomly-selected owners of steel buildings to assess their awareness of problems occurring in steel buildings, whether their building had been inspected by an engineer, and the state of damage, if any, found in their building. This preliminary survey was used to estimate the overall scope of damage to steel buildings and to help identify geographic areas where steel buildings were damaged. Based on results from this survey [3] and other more detailed information on ground motion intensity and structural damage, the Interim Guidelines recommended detailed inspection of steel buildings be conducted where peak ground motions exceeded 0.2g.

A third level of survey was carried out by engineers who had evaluated damaged steel frames [4,5]. Detailed information was obtained on 89 buildings regarding the types and locations of damage observed and the structural configuration, materials and detailing. This third survey was supplemented by even more detailed surveys of damage in 12 buildings selected for dynamic analyses. Precise comparison of predicted and observed damage was possible for these buildings.

Results were used to improve methods to select buildings for inspection, and to identify joints within a suspect building that should be inspected. For example, on average 70% of the floors of buildings surveyed had serious damage to at least one welded joint. Only 25% of the connections were found without damage. About 20% of the building frames had more than 40% of their connections damaged; in some instances, all connections at one or more floors were damaged.

Survey results also were used to assess methods for predicting damage. For instance, the data shows a correlation between damage and the area supported by a welded connection. This suggests that redundancy contributors to improved response, but other factors may be involved. Ground motion intensity was also found to correlate with damage, but limited data at high peak acceleration values makes precise interpretation difficult.

Similar interpretations showed that damage in low rise structures was more or less uniformly distributed over height, whereas tall buildings exhibited greater damage in the upper half. Results also show that damage tends to congregate. Thus, finding a severely damaged connection as part of an inspection should trigger inspection of other nearby connections.

DETAILED ANALYSES OF DAMAGED BUILDINGS

Twelve buildings damaged by the Northridge earthquake were selected for detailed analysis by consultants using elastic and nonlinear analysis programs. To support this effort, detailed investigations [6] of ground motion characteristics during the Northridge earthquake were conducted. This consisted of gathering available strong motion records in or near the subject buildings. In addition, a fault dislocation model was formulated, verified with available records and used to generate time history estimates at the sites of the case study buildings and elsewhere.

Buildings studied had heights from 2 to 17 stories, and were located from Santa Clarita, north of the epicenter, to Santa Monica, to the south. The analyses were intended to help identify the causes of the damage, as well as the ability of analytical methods and modeling assumptions to predict damage. For this reason, heavily damaged buildings were excluded. In two cases, buildings without damage were included. In four buildings, recordings of response during the earthquake were available and in three other cases, ambient vibration tests were performed.

The analysis results (see, for example, Refs. 7, 8 and 9) indicate that the case study buildings were very strong in comparison with the design forces incorporated in building codes. In many of the buildings the estimated response spectrum were nearly double those considered in current building codes (assuming elastic response, $R_w = 1$). Elastic analysis results showed that the most heavily damaged buildings were only stressed 2 to 3 times their capacities; in several cases, damaged buildings were predicted to remain essentially in the elastic range of response, suggesting that the buildings were 4 to 8 times stronger than required by code. The main reason for this over-strength appears to be the use of large-sized members to satisfy stringent code drift requirements.

Comparisons of damage survey data with results of elastic analyses of the buildings (using recorded and simulated Northridge earthquake records developed for the building sites [5]) show relatively poor correlation. Analyses suggest that the most heavily stressed joints are most likely to be damaged; however, the precise location and severity of damage was not reliably predicted by conventional elastic dynamic analyses. The 60% most highly stressed connections in a structure (relative to their capacities) have roughly equal chance of being damaged. Areas of low computed stress were also subject to damage. Thus, analysis may not be a good way of assessing the particular joints to inspect, though it may indicate floors that should be inspected. The reasons for differences between observed and computed behavior include the effects of initial defects and poor workmanship, and the limitations of current analytical methods and models. For instance, inclusion of slabs and panel zones had an important effect.

Elastic and inelastic dynamic analyses indicate that higher mode effects are important. As a result, equivalent lateral static force methods in the elastic range, and nonlinear push-over analyses have limited value for longer period structures. Similarly, the predicted distribution of damage in longer period structures is very sensitive to the ground motion considered. High velocity pulses in the ground motion records also resulted in substantial higher mode response, even for short structures. Additional analytical investigations were used to assess effects of structural modeling, member fracture, and structural configuration. These are reported in Ref. 10. Hypothetical buildings (especially shorter ones) subjected to severe shaking representative of the lightly populated areas of north of San Fernando Valley were most susceptible to collapse or severe damage.

Most design calculations are based on an assumption that plane sections remain plane during deformation. However, review of experimental data and results of finite element analyses suggests that this is far from true, with high local bending and shear deformations being induced in beam and column flanges. This is especially pronounced when plastic shearing deformations occur in the panel zone. Results demonstrated that these panel zone deformations were often very large. In such cases, the distribution of shear stress over the depth of the beam's web is not uniform, often concentrating the majority of the shear force in the highly stressed beam flanges. Compounding this situation is the fact that actual material properties are not uniform, and vary randomly from member to member and systematically with loading direction, section size, and welding procedures. Normal member to member variation of material properties may result in members stronger than the connecting weld, or a column that is weaker than the supported beam. As a result, the joint may have negligible inelastic deformation capacity, regardless of workmanship.

PRELIMINARY TEST PROGRAM

A total of 37 full size beam-to-column connections were tested as part of the Phase 1 investigation [9 and 11]. Twelve specimens (Fig. 1) were constructed in utilizing pre-Northridge details, half of the specimens had W36x150 beams and half had W30x99 beams. Fourteen-inch wide-flange sections were used as columns in both cases. Dual certified (f_y >50 ksi) steel was used. Slabs were not included. All specimens exhibited brittle appearing fractures; some fractured without any plastic deformation, while others deformed to a plastic rotation of 0.02 prior to fracturing.

The damaged specimens were repaired or upgraded. Repair consisted simply of rewelding the connections using high notch tough FCAW procedures; backing bars were removed, the root pass of the CJP weld on the beam flange to column flange connection was air-arc gouged and repaired with a fillet weld. This is the prevalent practice in repairs of damaged buildings in the Los Angeles area. Test results indicated that the repaired specimens, constructed with careful quality control, were able to retain their pre-damage strength and stiffness. Plastic rotation capacities were not significantly different from those achieved in the first tests. Thus, improved workmanship and materials did not significantly improve the inelastic performance of these details.

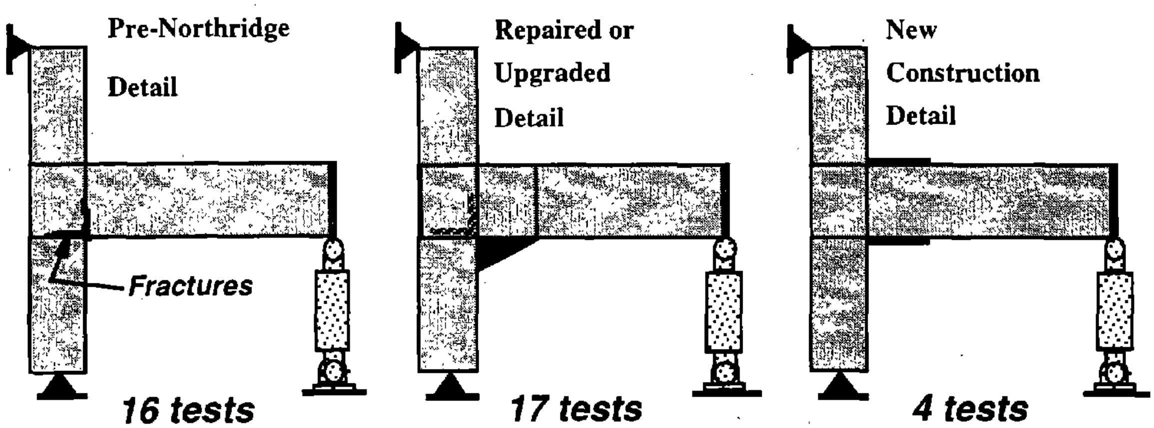

Figure 1 - Test Specimens Considered in Phase 1 Program

Some of the specimens were upgraded in an attempt to improve their plastic deformation capacity; inclined haunches were applied to one or both sides of the beam at its connection to the column. This detail moves the plastic hinge away from the face of the column. These tests supplemented earlier tests at the University of Texas [12] and elsewhere (see Refs. 1 and 13) which utilized trapezoidal- and rectangular-shaped cover plates, vertical fins, or side plates. Results for triangular haunches indicate that they are able to increase the plastic deformation capacity of the connection to a plastic rotation of at least 0.03. Inconsistent results have been obtained with cover plates.

Simple weldment specimens (Fig. 2) were tested to assess various weld procedures, initial defects, repair methods and loading rates [14]. These results clearly demonstrate the importance of quality welding and the greater reliability that can be achieved with high notch-tough weld consumables.

Tests were also conducted on a few details appropriate for new construction. These specimens utilized the same steel materials as for the previous beam to column tests, but utilized high notch tough weld wire. In addition, they included reinforcement of the end region to shift the plastic hinge region away from the face of the column. These specimens utilized horizontal cover plates and horizontal haunches. Generally superior behavior was obtained in these tests [11].

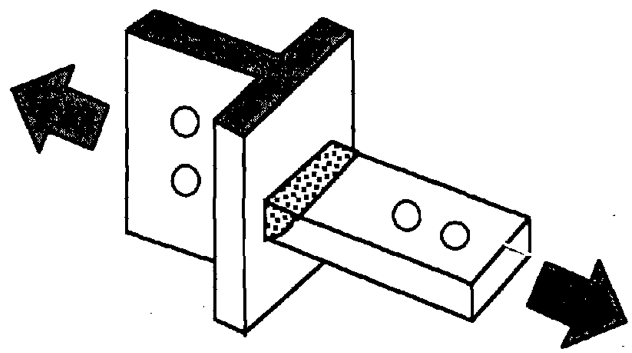

Fig. 2 Schematic illustration of simple weldment test specimen

OVERVIEW OF PHASE 2 EFFORTS

The Phase 1 ***Interim Guidelines*** provide the best answers within the current state-of-knowledge on what to do about welded steel moment frames. However, Phase 1 has also demonstrated the limitations of current knowledge. The substantial damage, including collapse, of many modern steel frame buildings in Kobe, and increasing reports of damage in the San Francisco Bay Area apparently due to the Loma Prieta earthquake, has heightened the appreciation worldwide of the need for developing reliable, practical and cost-effective solutions to this problem.

The Phase 2 effort addresses these solutions through eleven inter-related tasks spanning over 48 months. The detailed Work Plan for Phase 2 has been finalized through the efforts of the Technical Advisory Panels and the SAC management team, working in conjunction with FEMA and a Project Oversight Committee. Brief summaries of some of the technical investigation areas being undertaken to develop improved Seismic Design Criteria are presented below.

Performance of Steel Frame Buildings during Past Earthquakes - Various investigations are being undertaken to assess the performance of steel moment frame buildings in past earthquakes. In addition to the Northridge earthquake studies, information is being gathered related to the Kobe, Landers/Big Bear, Loma Prieta, Whittier Narrows, and other earthquakes. Results will be interpreted to help assess damage screening and inspection criteria, identify details and other structural features associated with the presence or absence of damage, evaluate the accuracy of analytical methods, and assess the economic, social and other impacts of damage.

Materials and Fracture - This task examines the mechanical properties of steel materials in commercially-available structural members, including new materials just coming on the market. It also identifies the influence of various factors on the behavior of simple, fracture critical welded joints such as the orientation, history and rate of loading, the strength and notch toughness of base materials, joint restraint, and local details. Material characteristics required to develop proposed connection details will also be identified.

Joining and Nondestructive Testing - A variety of investigations are undertaken in coordination with investigations related to Materials and Fracture and Connection Performance. These aim at understanding the factors (e.g., welding consumables, procedures, and the relative strengths of the weld metal and base metal) that control behavior, establishing the sensitivity of ultrasonic testing techniques, assessing promising new NDE procedures, and developing criteria for welding and inspection. Bolted and partially restrained joints are also studied.

Connection Performance - Detailed finite element and other analyses are being utilized to devise methods for predicting the deformation and strength capacities of connections and to develop simplified analytical methods suitable for design practice. These analyses will be closely coordinated with the connection test program. Tests will be conducted initially to assess parameters that control the behavior of promising new details as well as of commonly used pre-Northridge and current designs; later tests will be used to validate the details and design methods to be incorporated in the Seismic Design Criteria. Tests will include single and double sided beam to column connections, with and without slabs. In addition to welded steel moment connections, simple, bolted and partially restrained connections will be studied.

System Performance - Focused investigations are underway to assess the effect of various structural and ground motion parameters on global and local demands. Hypothetical buildings having 3, 9 and 20 stories, located in regions of relatively high, moderate and low seismicity, are used as the basis of these studies. Different computer programs and modeling approaches are being utilized to study the effect on seismic demands of ground motion intensity and dynamic characteristics, and structural configuration, proportioning and modeling, as well as of the deterioration of hysteretic characteristics due to local buckling, brittle fracture, and so on. Also, the safety and reliability of steel moment-resisting frame systems will be evaluated considering the possible occurrence of brittle fractures. Potential benefits of alternative framing systems having partially restrained, bolted or energy dissipative connections are being investigated.

Performance Prediction and Evaluation - Results of the investigations on seismic demands are being synthesized and interpreted along results of studies on the capacities of various details and connections to achieve a consistent set of performance-based design and analysis procedures for steel moment frame structures. These are directed at the evaluation of existing steel buildings as well as the design of new ones. Analysis and modeling simplifications suitable for design are being assessed, and special procedures for regions of lower seismicity are being examined.

Economic, Social and Political Issues - A variety of activities are being undertaken to assess the practicability of the Design Criteria and to assess the potential economic, social and political impacts of their implementation. These activities include trial applications and economic and performance assessments of buildings designed using various procedures and criteria, and identification of other barriers to effective implementation of the final Seismic Design Criteria.

CONCLUDING REMARKS

While the Interim Guidelines represent current U.S. thinking on the proper evaluation, inspection and repair of WSMF buildings, there are clearly many uncertainties and unresolved questions. In Phase 2 of the FEMA/SAC Steel Program additional research and testing will more clearly define the parameters controlling the performance of connections and systems, and develop and verify reliable and cost effective procedures for design of new moment frame buildings and for the evaluation and rehabilitation of existing ones.

ACKNOWLEDGMENTS

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Steel MR Connection Design Critically Reviewed

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SUMMARY

Design of Moment Resisting (MR) connections of pre-1994 Northridge earthquake are critically examined from engineering mechanics and statistical points of view. For this purpose, histograms for tensile mill tests of ASTM A36, A572 Grade 50 and Dual Grade steels are brought in. Then the fundamental limitations of the conventional tensile test are discussed. Kuwamura/Kato statistical results on the dependence of plastic joint location within a steel frame on the ductility are then cited. T-S. Yang's detailed finite element results bring into question the universally employed formula VQ/It for calculating shear in the MR connection. Based on the above background, experimental results performed at Berkeley on the pre-Northridge earthquake MR connections are discussed. A brief description of the successful Berkeley results with Dog Bone tests and the reason why they behave well completes the paper.

HISTORICAL BACKGROUND

For seismic application for years the structural engineers placed great confidence in steel framing. The resulting relatively lighter weight of such systems was known to attract smaller earthquake forces, and the reliability and simplicity of connections for such construction was widely accepted. Therefore the extensive damage due to a moderately large 7 second Northridge earthquake in 1994 sent a shock through the design profession. As it turned out this was the most costly earthquake in the U.S. history.

Historically much of the advance in the design of steel structures can be justly credited to the Lehigh University with its active research work in this area, and much good advice from Ted Higgins the founder of the AISC around 1923. However, the research at Lehigh as well as everywhere else was directed at largely gravity loadings, and capacities of members and connections were studied under monotonically applied forces.

The first cyclically applied loadings were initiated at Berkeley. After a modest start by Bertero and Popov, only around 1967 at the suggestion of the late "Pete" Kellam, a San Francisco structural engineer, an AISI funded project was started at Berkeley. The project dealt with determining the behavior of 7 ft (2.13 m) long cantilevers and their connections rigidly held by columns. A select committee supervised and gave much good advice on the project[1]. The results of this work reported in the AISI Bulletin No. 21 (Popov and Stephen, 1972) gained wide acceptance, and it is instructive to examine some of the results from this work.

The current stress-strain diagrams for steel are significantly different for A36 material from the earlier ones (see **Figure 1**). Note that one of the A36 (250 MPa) steel just reached the specified yield and F_u/F_y was 1.72. The companion specimen had a yield plateau at 45 ksi (310 MPa) and an F_u/F_y of 1.56. The present steels have average values of F_u/F_y for A36 steel of 1.42. This ratio shows that the new steels have a smaller margin of reserve strength beyond yield.

All connections had the beam flanges attached to the rigid columns with complete penetration welds. Most of the webs were bolted to the shear plates. In two of the eight specimens, the webs were fillet welded to the shear plates. The all welded connections behaved somewhat better than those with bolted details. One of the poorer hysteresis loops for a bolted web connection is shown in **Figure 2** are not sufficiently large, having only about 1.2% maximum plastic hinge rotation. Whereas this is not good, it is much better than many subsequent post-Northridge earthquake tests. The concept of hinge plastic rotation was not appreciated at the time of these tests.

Another series of Berkeley tests completed in 1984 of subjecting a panel zone of an axially loaded column to bending moments from beams on two sides, **Figure 3**, were particularly meaningful. These experiments were made to demonstrate the need for continuity plates, see **Figures 4** and **5**. It would appear that a major city in California was accepting doubler plates on the column panel zone in lieu of the continuity plates. Note that the plastic rotation of a joint with continuity plates was 2.8%, which for the time the tests were made were considered completely adequate.

[1] Bertero, Collin, Daniels, Degenkolb, Eberhart, Gilligan, Johnston, Napper, Pinkham, and Viest

FIGURE 1
STEEL STRESS-STRAIN DIAGRAMS FOR 1969 TESTS

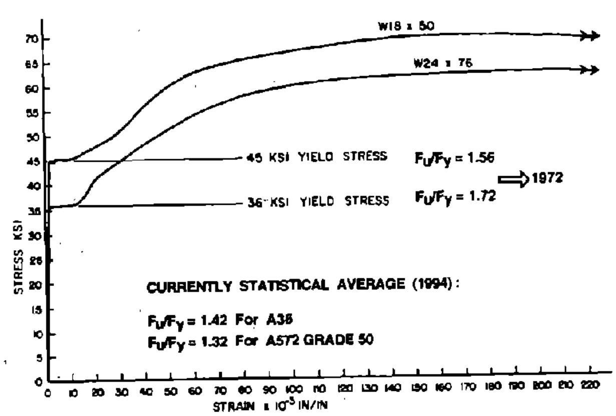


FIGURE 2
HYSTERESIS LOOPS FOR W24×76 BEAM WITH 7-1" WEB-BOLTS

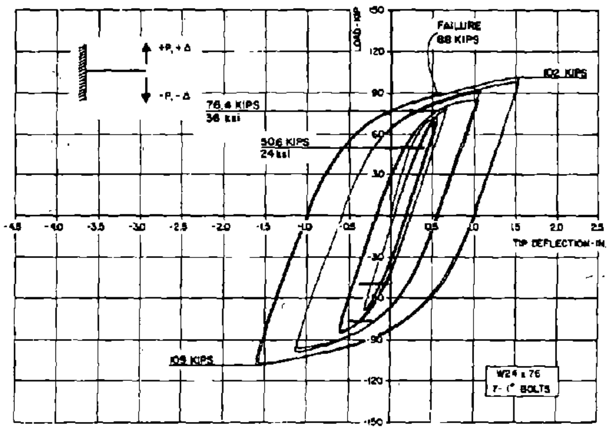


FIGURE 3
EXPERIMENTAL SETUP FOR COLUMN PANEL ZONE CYCLIC LOADING. THE BEAM END LOADS WERE APPLIED IN OPPOSITE DIRECTIONS

FIGURE 4
HYSTERESIS LOOPS FOR W18×40 BEAM WELDED TO COLUMN WITH NO CONTINUITY PLATE

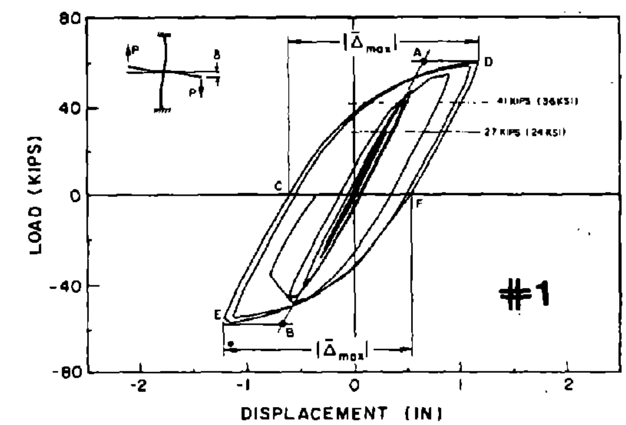


FIGURE 5
HYSTERESIS LOOPS FOR W18×40 BEAM WELDED TO COLUMN WITH CONTINUITY PLATE

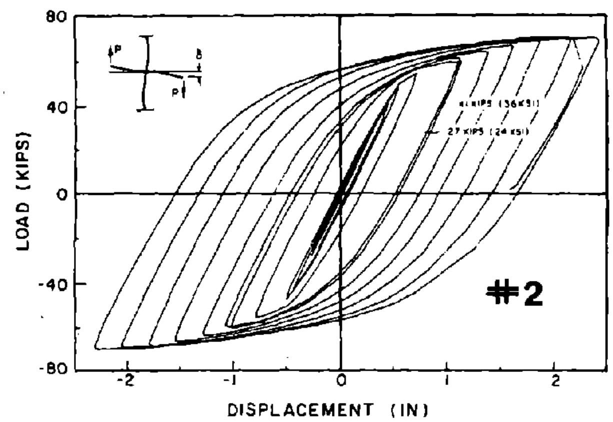


FIGURE 6
YIELD POINT VS. WEB THICKNESS

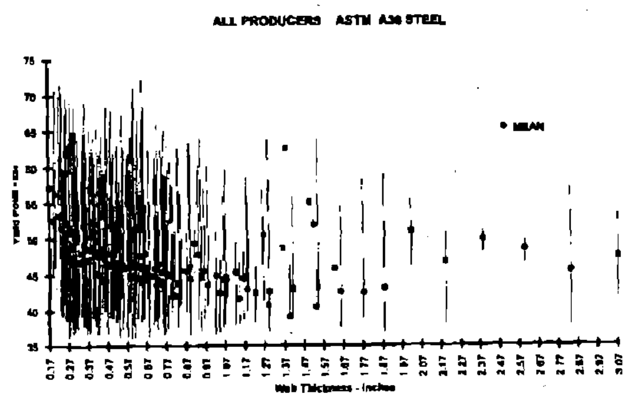
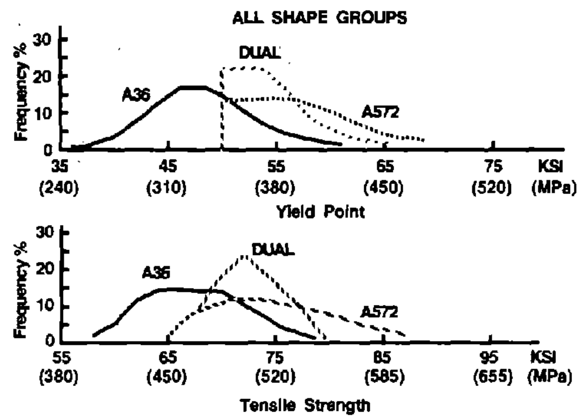


FIGURE 7
ALL SHAPE GROUPS HISTOGRAMS

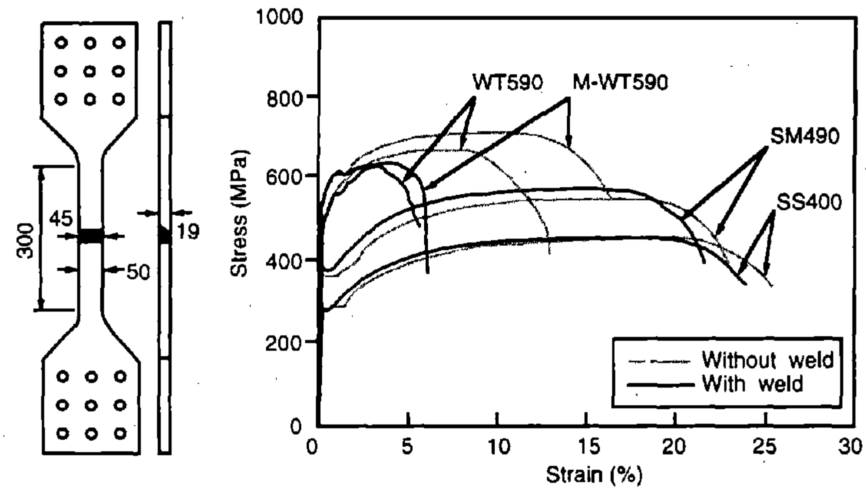


THE NEW STEELS AND THEIR WELDS

In recent years, significant changes in steel making and rolling practices have occurred. Now some of the mills use electric furnaces to melt steel from a 100% scrap charges. The other mills in addition to the molten iron from blast furnaces add a 30% scrap charge. This generates considerable variability in the steels.

Frank (1994) on the basis of 57,930 certified mill tests generated histograms and summary statistics for various grades of steels. The data were obtained from the webs of various wide-flange shapes. This information is shown in **Figure 6**. Note the variation in yield point within each web thickness. The mean for each thickness is designated by a black square. These mean values are reported in two histograms in **Figure 7**, one is for the yield point, the other is for tensile strength. Several groups of such histograms are given in the Frank report for different grades of steel. The yield strength as well as the tensile strength histograms for grade A36 resemble Gaussian distribution.

FIGURE 8
STRESS-STRAIN DIAGRAM FOR THE WELDED AND UN-WELDED TEN SPECIMENS (COURTESY IWATA)



However the coefficient of variation (COV) appears large. The situation is worse with A572 steel, especially for the yield point. The same ill defined situation can be observed for the dual grade steel. A designer faced with a hypothetical allowable stress, in reality has no knowledge regarding the actual safety of a structure. At the cut-off point of 50 ksi (345 MPa) the reality and the design calculations are not in agreement.

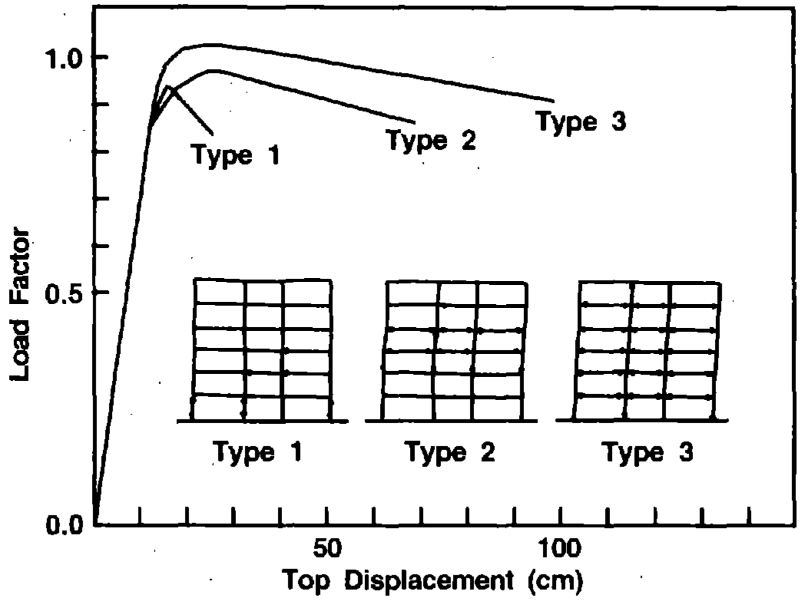
Welding is a subject in itself, and although very important, is beyond the scope of this paper. However, its effect on the stress-strain behavior of steel can be noted.

Iwata and his associates (Wada et al., 1996) at the Tokyo Institute of Technology carried out some very interesting experiments. By using specimens of the type shown in **Figure 8**, two kind of specimens were tested. In the one group the shank of the specimen was continuous, in the other, the specimens were welded in the middle as shown in the figure and machined to the original size. These specimens were subjected to cyclic loading in four steps with four cycles in each step reaching 1% strain. For the steels SM490 and SS400 approximately corresponding to A36 Grade steel, essentially no difference can be noted for specimens with and without a weld. The situation is quite different for the stronger steels WT590 and M-WT590 corresponding to 50 grade steel. This bids bad omen to the higher strength steels with welding!

SOME ANALYTICAL CONSIDERATIONS

In an excellent paper by Kuwamura and Kato (1989) statistical analysis using Monte Carlo simulation, three bay-six story frame was studied. The COVs for random yield stresses were set at 0.10, 0.05, and 0.025. The column overdesign factors (COFs) had the assigned values of 1.1 through 2.0. The correlation coefficient among the beams was taken from 0.0, 0.7 and 0.9. Using these assumptions, for each case, 200 sets of random numbers corresponding to 200 different six-story frames were analyzed. These solutions yielded three different cases or types of collapse modes, see **Figure 9**.

FIGURE 9
LOAD-DISPLACEMENT RELATIONS AND THEIR FAILURE MECHANISMS



For Type 1, corresponding to the first story collapse, the frame shows very poor ductility. As the number of failure stories increases, so also does the ductility of the frames. The best case, Type 3, corresponds to formation of plastic hinges in all beams at the column faces, with the exception of those at the roof. To attain the best kinematic displacement, plastic hinges must also form at all column bases. These conditions are commonly assumed in routine calculation by the analysts. The authors of this paper strongly recommend weak-beam-strong-columns to avoid an early collapse mechanism. This assertion can be put into the form of an equation

$$\frac{\sum Z_c(F_{yc} - P_{uc}/A_g)}{\sum Z_b F_{yb}} = 1.1 \text{ to } 2.0 \tag{1}$$

where Z_b and Z_c are the plastic modulus of the beam and column, respectively, and F_{yb} and F_{yc} are the yield stress of beam and column, respectively. P_{uc} and A_g are the column axial force and gross area, respectively.

This equation is specified in the AISC Seismic Provisions (1992), except that the right hand side of the equation is **set unconservatively equal to unity**.

EXPERIMENTAL CORROBORATION

Several experiments were supervised by the authors under the SAC[2] Program. Four specimens were tested as built, three of them were retested after repair (Popov et al., 1996). Here attention is

[2]SAC is an acronym for Structural Engineers of California, Applied Technology Council, and California Universities for Research in Earthquake Engineering.

FIGURE 10
SAC PN1 AFTER TEST

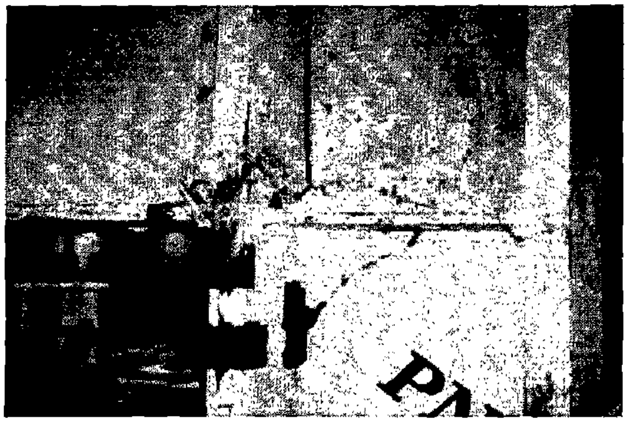
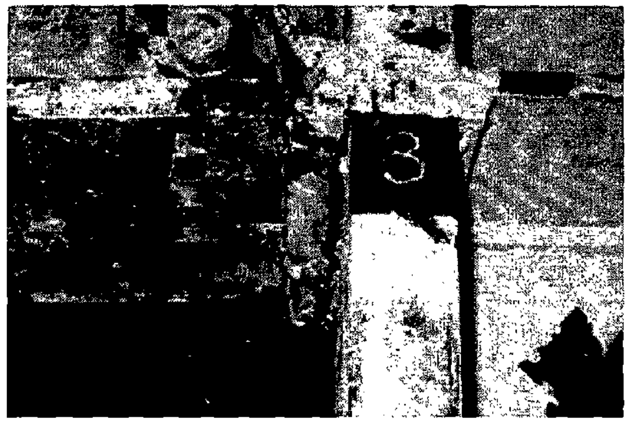


FIGURE 11
SAC PN3 AFTER TEST



confined only to two of the experiments. One is concerned with Specimen PN1 where by mistake in the fabricating shop, a W36×150 (W920×223) A572 Grade 50 cantilever beam was substituted for an A36 grade beam. This resulted in a beam having an F_y=62.6 ksi (432 MPa), and the W14×257 (W360×382) A572 Grade 50 column having an F_y=53.5 ksi (370 MPa). At F_u both members had approximately an equal strength of 74 ksi (510 MPa). In a cyclic experiment at a little less than 1% of plastic rotation the specimen precipitously fractured through the column, see **Figure 10**. A similar specimen with identical detail and material fractured in the same manner. These were the first cases encountered in the SAC program. It would appear that these were the only ones observed in any laboratory. Therefore it seems reasonable to speculate as to why some columns cracked as a result of the Northridge earthquake. It is plausible that in such cases, quite legally, the fabricator used dual grade steel. In such cases A36 steel may be very near or exceed the 50 ksi (345 MPa) limit and the 50 grade column steel my have barely qualified.

These results strongly corroborate the conclusions reached by Kuwamura and Kato (1989), and Eq. 1 is recommended for the adoption in the code.

The SAC specimen PN3, having a correct distribution of the material, i.e., a W36×150 (W920×223) A36 beam and W14×257 (W360×382) A572 Grade 50 column, behaved differently. Typical of pre-Northridge designed connections, the fracture occurred at the column face through the beam or weld, see **Figure 11**. Regarding this typical fracture some further comments are in order. As emphasized early by Kirkaldy (1862), Ludwik and Scheu (1923), Timoshenko (1930) and many other researchers, at restrained locations the stresses are greatly increased. As a result, under Timoshenko's inspiration MacGregor (1931), carried out a series of tests with different size grooves in the tension specimens. The results of this work are summarized in **Figures 12 and 13**. Note that the bar with the narrowest groove developed the largest stress, but very limited ductility. As the size of the grooves increased, the strength of the maximum strength of the specimens decreased, and their ductility increased. Only when the bar shank become long, does the stress-strain diagram corresponds to the one normally cited to illustrate steel ductility. The above condition can be likened to the situation of a joint between a beam flange and the column. The flanges of the column and the beam effectively restrain deformation. Lastly it is again well to return to the theory. A refined

FIGURE 12
GROOVED BARS TESTED

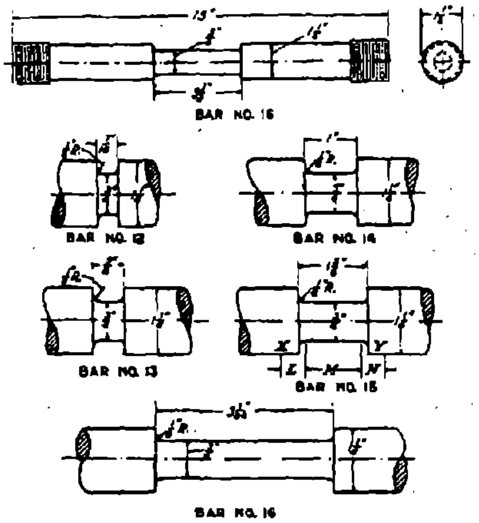
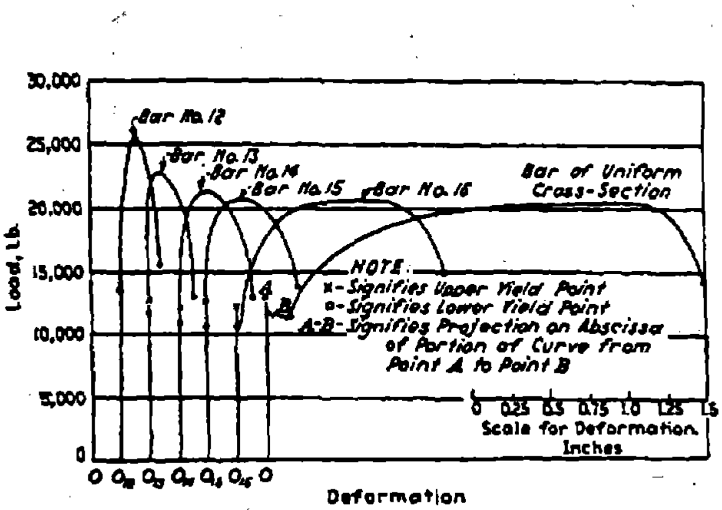


FIGURE 13
LOAD-DEFORMATION DIAGRAMS



nonlinear finite element analysis of von Mises stresses is shown in **Figures 14 and 15** caused by the application of a vertical force 134.5 in (3.4 m) from the face of the column (Yang and Popov, 1995). Figure 14 clearly shows a band of high stresses in the beam next to the column, with a peak in the middle. The pattern of stresses shown in Figure 15 definitely shows that next to the support, i.e., at the column the stresses are very different from those predicted by the elementary mechanics. This places under serious doubt the validity of the universally used assumption of "plane section remains plane" during bending at the supports. The basic shear formula $\tau = VQ/It$ needs to be thoroughly re-examined.

MR CONNECTION FRAMING WITH DOG BONES

Both in the field and the laboratory the conventional steel moment resisting connections behaved poorly. It appears to be very difficult to develop the ultimate capacity of a beam framing into a column. The alternative is to reduce the bending capacity of a beam next to the column such that the strength of a conventional connection would be adequate. An example as to how the beam flanges can be reduced to achieve this purpose is shown in **Figure 16**. Suggested within this group of the authors in 1994, and re-invented several times, it is believed to be the best among the alternative shaping of the beam flanges. Cuts of the beam flanges to a large radius during loading causes little local stress concentration. This approach was successfully tested at several laboratories. The purpose of this discussion is to explain why it works.

The analysis using plane stress flange elements is shown in **Figure 18**, whereas useful in showing the general longitudinal stress distribution, is not detailed enough for the problem at hand. What is needed is an analysis consisting of several layers of brick elements in the beam flanges. This is shown in **Figure 19**, not shown are the stresses that exist in the other two orthogonal directions. These have been verified, but for lack of space are not reported. The answer why the Dog Bone can be greatly deformed can be seen from a qualitative diagram in **Figure 17**. When there are

FIGURE 14
STRESS CONTOURS – BOTTOM VIEW

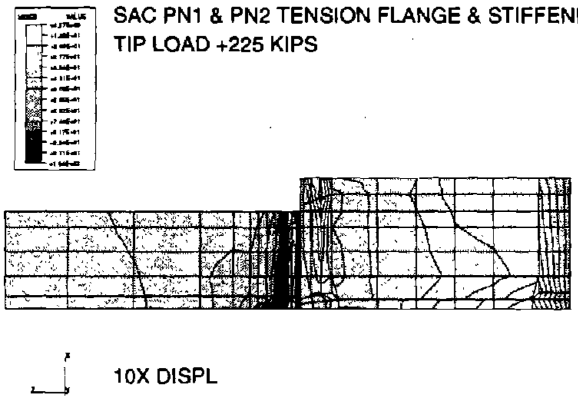
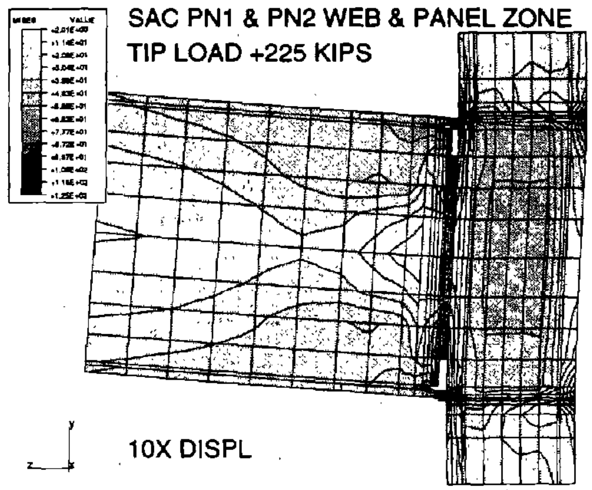


FIGURE 15
STRESS CONTOURS – SIDE VIEW



orthogonal stresses, the stress and strain capability of the material greatly increases. In a two-dimensional space usually used in elementary plasticity, this does not occur. As can be seen from **Figure 17**, a large increase in stress and the corresponding strain is possible in 3D.

DEDICATION

This paper is respectfully dedicated to Professor Vitelmo V. Bertero, a true academician and an investigator trying to discover the intricacies of structural behavior.

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FIGURE 16
DOG BONE FINITE ELEMENT MESH

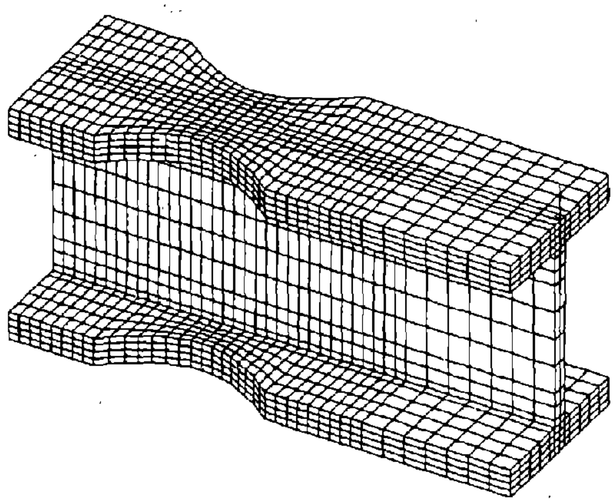


FIGURE 17
YIELD SURFACE OF DOG BONE

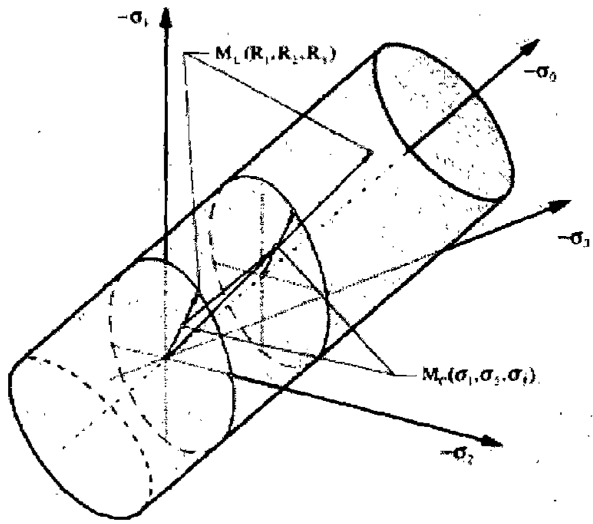


FIGURE 18
AXIAL STRESS CONTOURS (2D)

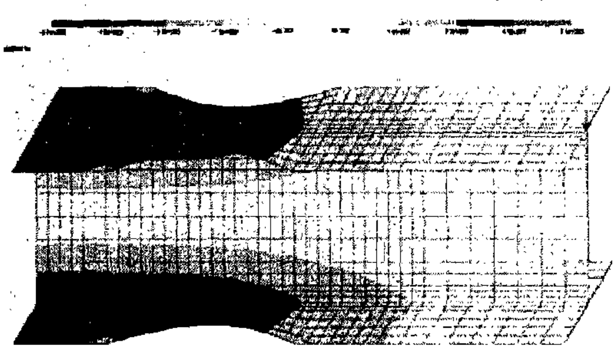
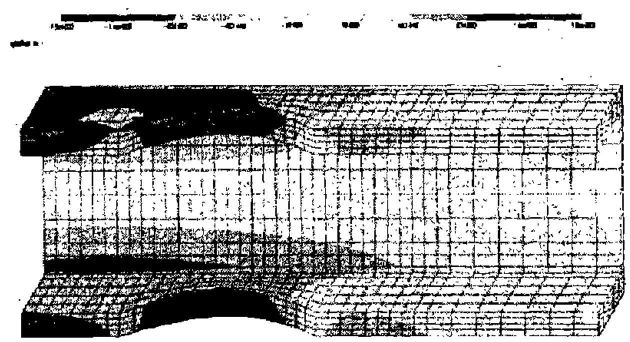


FIGURE 19
AXIAL STRESS CONTOURS (3D)



A Welded Moment Connection for Low Rise Steel Frames

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SUMMARY

An experimental study has evaluated the strength and rotation that can be supplied by a repair/retrofit detail for welded beam to column moment connections. The connection detail uses a vertical triangular plate (fin) welded to the top and bottom flanges of the beam in the plane of the beam web. Plastic rotations of 2.7% were developed under displacement controlled cyclic load. Based on this behavior, the detail has been used for a new, two story steel structure. Using two of the stronger earthquake records obtained during the Northridge earthquake, nonlinear dynamic analyses of the building were conducted to estimate the strength and rotation demands of these two motions. Results indicate that the maximum rotation demands are just equal to the maximum rotation supplied by the connection detail, indicating that the design is adequate.

INTRODUCTION

In order to achieve a successful, earthquake resistant design, it is necessary that the strength and deformation capacity supplied by the structural components be greater than the strength and deformation demands placed on the components by the earthquake ground motions. The cracking in welded beam to column connections of modern steel buildings which was discovered following the Northridge earthquake observations by Bertero, et al. (1994), indicate that this basic design principle was not satisfied. In most cases, the strength was probably adequate, however, the welds of critical moment connections cracked with little or no plastic deformation indicating that the deformation demand was much greater than that which could be supplied with the current connection configuration and fabrication.

The most common type of cracking occurred in the welds at the bottom beam flange and generally started at the center and propagated outward. This occurs because there is a high stress concentration at this location, the weld is discontinuous and the web cope hole is a source of crack initiation. Web cope holes are necessary to accommodate the welding of the top and bottom beam flanges, however, the geometry and quality of these holes varies considerably. They are usually flame cut and often are not ground smooth, giving rise to a rough surface which is ideal for crack formation. This is particularly true when the beam flanges begin to buckle under cyclic loading and plastic hinging. Therefore, a successful design procedure must neutralize the negative effect of the web cope by lowering the stress concentrations in this critical region.

The need to estimate the inelastic deformation demands of earthquake ground motions on the welded connections of steel moment frames makes the use of inelastic dynamic response analysis an essential part of the design process. In order to have confidence that these estimated demands can be satisfied by the connection detail, it is necessary to conduct controlled cyclic load tests and to measure the strength and rotation that can be supplied by the connection. In this manner the basic principle of supply greater than demand can be satisfied with a high level of confidence.

The application of this basic procedure for the design of a two story steel building is discussed in the following sections. In this case, the testing was done first as part of a program of investigation on ways to repair/retrofit steel frames following the Northridge earthquake as described by Anderson, et al. (1996,p.768). The results for one of these procedures was so encouraging that it was incorporated in the design of a new, low rise steel building.

EXPERIMENTAL PROGRAM

In order to evaluate the strength and deformation that can be supplied by the modified moment connection, a cyclic load test was conducted on a full scale test specimen representative of the smaller steel sections used for low rise buildings. The basic test specimen, prior to retrofit, consisted of a W21x68 beam welded to a W12x106 column using a standard weld detail representative of welded moment connections prior to the Northridge earthquake. The specimen was tested with the column vertical as shown in **Fig. 1**. A constant axial compression load of 20 kips was applied to the 9 1/2 foot column and a cyclic load under displacement control was applied at the end of a 6 foot beam. All columns had continuity plates across the column web in addition to a web doubler plate on one side of the column web, extending eight inches above the top continuity plate and eight inches below the bottom continuity plate.

The steel for the beam was specified as A36 although the yield strength obtained from coupon tests was 47.5 ksi. The steel for the column was A570 Grade 50 with a coupon yield stress of 56 ksi. The loading sequence followed the general direction of the protocol suggested by Krawinkler (1992), however, changes were made to accommodate the test configuration used in this study and to assure that enough intermediate data points would be obtained prior to failure.

The repair/retrofit detail incorporates a vertical triangular plate (fin) above the top beam flange and below the bottom beam flange, in line with the beam web. A detail of the fin plate is shown in **Fig. 2**. This diverts some of the force in the beam flange around the connection of the beam flange to the column flange. Stresses in the welds are reduced by a combination of the additional weld material and by the reduced force in the beam flange due to the increase in the reactive moment arm. Initial tests indicated that it is desirable to limit the weld stresses in the fin at the face of the column by drilling hole in the fin. For the size of fin used for this specimen, a one and 1 1/2 inch diameter hole worked well. This moves the critical section of the fin from the face of the column to a section through the hole, causing the fin to yield and thereby limit the stresses transmitted to the welds at the column face. Hence the fin acts as a structural fuse to limit the weld stresses. Two sides of the fin are beveled to accommodate full penetration welds to the column flange and beam flange.

EXPERIMENTAL RESULTS

The specimen with vertical fins and web doubler sustained 16 displacement cycles ranging from 1/2 inch to 2 3/4 inches as shown by the displacement history, **Fig. 3**. The maximum displacement represents a total rotation of 4.1%. The history of the force at the beam tip, shown in **Fig. 4**, indicates that during the last two cycles the specimen is unloading by approximately 26%. At this time both the top and bottom beam flanges had buckled along with the beam web. A small crack developed at the toe of the bottom fin due to the prying action of the buckled beam flange, however, it did not propagate or cause any loss of capacity. The plastic hinge region, shown in **Fig. 5**, developed 17 inches from the face of the column, thereby eliminating problems with the web cope holes. Yielding at the section through the holes in the fin can be seen by the flaking of the whitewash. The moment versus plastic rotation curve, shown in **Fig. 6**, indicates a plastic rotation of 2.6%. The corresponding elastic rotation was approximately 1.5% giving a total rotation of 4.1%. It should be noted that the addition of the fin increases the moment capacity of the connection by approximately 20%. The test was stopped due to severe deformation in the beam and concern for damaging the testing equipment.

BUILDING SYSTEM

The building is a two story steel structure with each story 14 feet in height and a plan which is 255 feet by 79 feet. The second floor and roof consist of 2 1/2 inches of light weight concrete over metal deck and spread footings are used for the foundation. An isometric view of the steel skeleton is shown in **Fig. 7**. Lateral resistance is provided by two moment resistant perimeter frames in the longitudinal direction and by six moment frames in the transverse direction, two at the ends of the building and four on the interior as shown in **Fig. 8**. The detail of a typical moment connection, including the fins, is shown in **Fig. 9**. The fins were welded to the beams in the shop and then welded to the column flange once they were in position in the frame. A picture of a typical connection prior to welding to the column flange is shown in **Fig. 10**. When the floor decking is in place and the concrete floor is poured, the top fin will be completely covered.

The design base shear for earthquake loading is specified as 0.14W on the structural drawings where W is the effective dead load of the structure, estimated to be 2811 kips. This results in a code design base shear of 394 kips based on working stresses and an ultimate base shear of approximately 550 kips.

DYNAMIC RESPONSE ANALYSES

Results of modal analyses, conducted using the ETABS program, Habibulah, (1992), indicate that the first mode is a translational mode in the transverse direction (E-W) and has a period of 0.77 seconds. The second mode has an almost identical period and is a translational mode in the longitudinal direction (N-S). The third mode is a rotational mode having a period of 0.71 seconds.

In order to estimate the strength and displacement demands on the critical structural elements, nonlinear dynamic response analyses were conducted for two of the stronger earthquake ground motions recorded during the Northridge earthquake. These two records are the one recorded at the Newhall Fire Station and the one recorded at the Sylmar County Hospital. Preliminary elastic response analyses indicated that the north-south (N-S) component of both records was the stronger component and therefore the N-S component of each record was used for the nonlinear analyses.

Due to the symmetry of the lateral force system, an in-house two dimensional nonlinear analysis program was used to estimate the strength and deformation demands. In the N-S direction, the response of one of the perimeter frames, shown in **Fig. 11**, is evaluated. In the E-W direction, one of the end frames and two of the interior transverse frames are linked together by rigid links representing the floor diaphragm as shown in **Fig. 12**.

The envelope of maximum interstory drifts determined by the nonlinear analysis for the transverse (E-W) frames and the longitudinal (N-S) frames is shown in **Fig. 13**. It can be seen that drifts in both the N-S and E-W frames are larger under the Newhall ground motion with the maximum drift reaching 4% in the N-S frame. For the Sylmar motion, all drifts are less than 2.8%.

The estimated maximum rotation demands for the girders are shown in **Fig. 14**. The maximum demands occur under the Newhall (N-S) ground motion and have a maximum value of 2.7% in the second story level which is approximately equal to the rotation supplied by the connection in the test (2.6%) although the test was stopped before complete failure of the connection occurred. Based on the limited test results, it can be concluded that the connections of the lateral force system used in this building can supply the rotation demands of the Newhall and Sylmar ground motions.

MATERIAL YIELD STRENGTH

It is well recognized that the rotation demand will depend on the characteristics of the earthquake ground motion. For this reason an ensemble of possible ground motions at the building site need to be considered in developing the design envelope of maximum rotation demands. Another variable that needs to be considered is the yield strengths of the connection members. The actual yield strengths may be considerably higher than the specified minimum values. While this increase will have a positive effect on an elastic response analysis (reduced stress ratios) the effect on a nonlinear response analysis is not certain and must be considered as part of the analysis process.

As part of this study, four combinations of girder yield stress/column yield stress were considered. The 36/36 combination represents a lower bound using the nominal value for A36 steel. A combination of 36/50 represents the nominal values of the steel members used in the building. The 42/50 combination represents the values used by the structural engineers for the building design and the 47/56 combination represents the actual yield values for the test specimen. The Newhall (N-S) ground motion is used as the input ground motion for all comparisons.

The effect on the interstory drift of the N-S frames, shown in **Fig. 15**, is substantial. The increase in the drift is 48% in the second story and 75% in the first story with the lower drift obtained using the lower strength steel and the higher drift a result of the higher strengths. The effect of the yield strength on the plastic rotation demands of the girders, shown in **Fig. 16**, is not as dramatic, but still substantial. As before, the lower rotation demand occurs with the lower strength steel and an increase of as much as 30% occurs with the higher strength materials.

The effect of the yield strengths on the drift and rotation demands on the E-W frames is shown in **Figs. 17 and 18** respectively. In this direction, the changes in the response due to the changes in yield value is not as significant as in the previous case with the maximum increase in drift demand being approximately 15% and the maximum increase in rotation demand being 24%. Since all of the plastic rotation demands are less than 2.6%, the connection detail is considered to be adequate.
It should also be emphasized that these results are not general, but depend on the interaction of the input ground motion and the material yield strengths.

CONCLUSIONS

In order for an earthquake resistant design to be successful, the strength and deformation capacities supplied by the structural components must be greater than the corresponding demands placed on the building by representative earthquake ground motions. A seismic design procedure for steel frame buildings is presented which utilizes the results of cyclic load tests on full scale connections to quantify the strength and deformation which can be supplied. Furthermore, the results of nonlinear dynamic analyses are used to estimate the strength and deformation that will be demanded by the selected earthquake ground motions.

Results from a case study, two story steel building indicate that the addition of a vertical, triangular fin to the top and bottom beam flanges can reduce the stress concentrations in the connection and produce plastic rotations in excess of 2.7%. Additional testing will be required for extension of the vertical fin concept to taller structures with larger member sizes. The nonlinear dynamic analyses indicate that the maximum plastic rotation demands for the two ground motions considered in this study are just equal to those supplied by the connection detail, indicating a successful seismic design.

Since the inelastic response is sensitive to the input ground motion, it is necessary to consider an ensemble of earthquake ground motions that are representative of those that might be experienced at the building site. It is also noted that the actual yield strength of the material needs to be specified within certain bounds since variations in this parameter can influence the estimate of plastic rotation demand. Without such a limit, it will be necessary to consider several possible combinations of the yield strength of the beams and the yield strength of the columns.

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ACKNOWLEDGMENTS

The experimental study was funded in part by grants from the National Science Foundation, the American Institute of Steel Construction, the Structural Steel Education Council and the Structural Shape Producers Council. The author also acknowledges the contributions of Dr. Gregg Brandow, Mr. Don Strand and Mr. Peter Maranian of Brandow & Johnston Associates to various parts of this study. Ms Xiaojing Duan of USC contributed to several aspects of the study.

Fig. 1. TEST SETUP

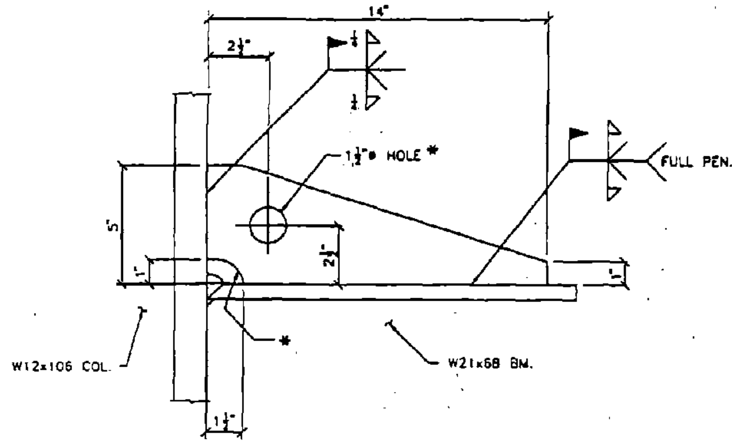

Fig. 2. FIN DETAIL

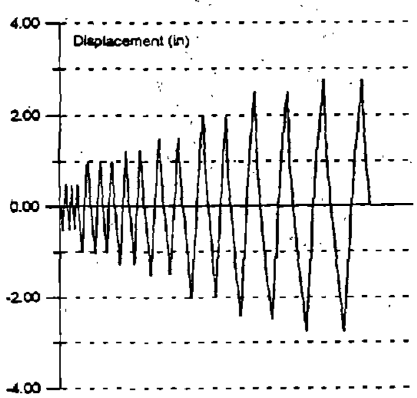

Fig. 3. DISPLACEMENT HISTORY

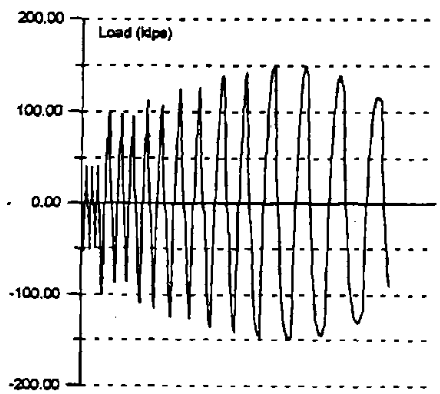

Fig. 4. LOAD HISTORY

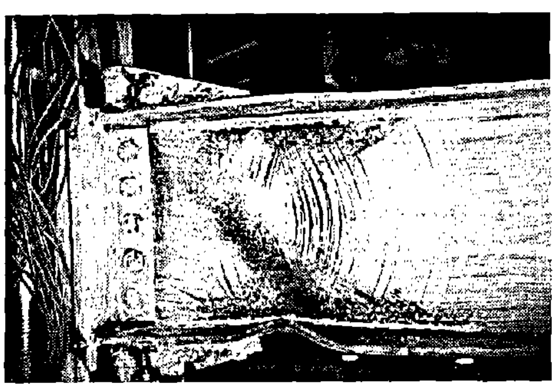

Fig. 5. PLASTIC HINGE

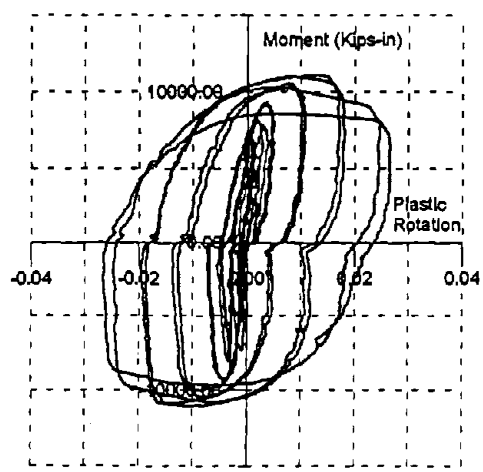

Fig. 6. CONNECTION HYSTERESIS

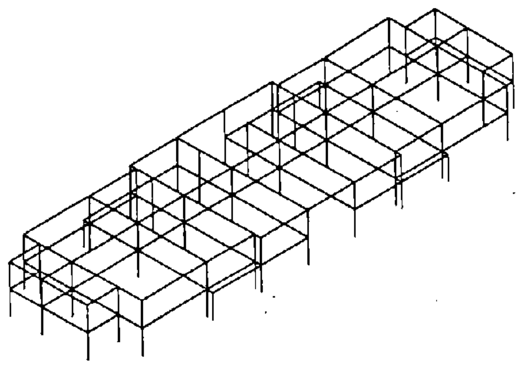

Fig. 7. STEEL FRAMING

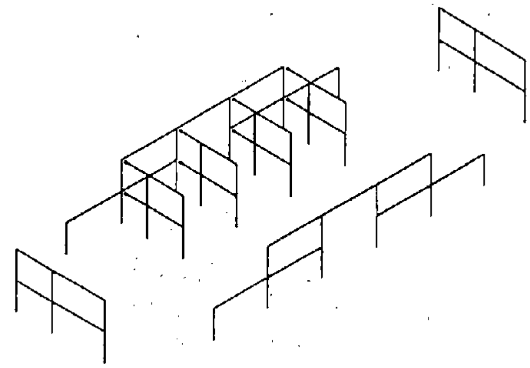

Fig. 8. LATERAL FRAMING

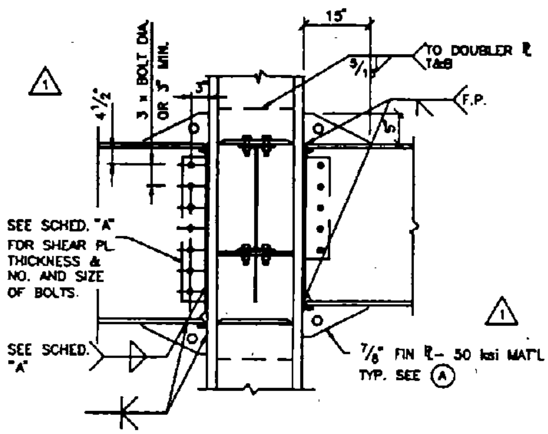

Fig. 9. CONNECTION DETAIL

Fig. 10. ACTUAL CONNECTION

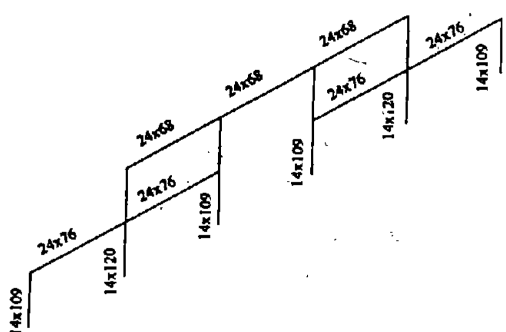

Fig. 11 NONLINEAR MODEL (N-S)

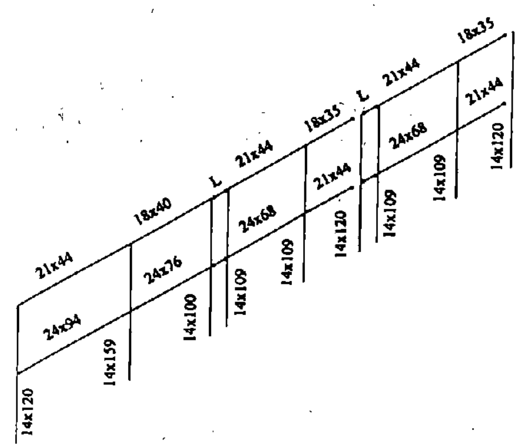

Fig. 12. NONLINEAR MODEL (E-W)

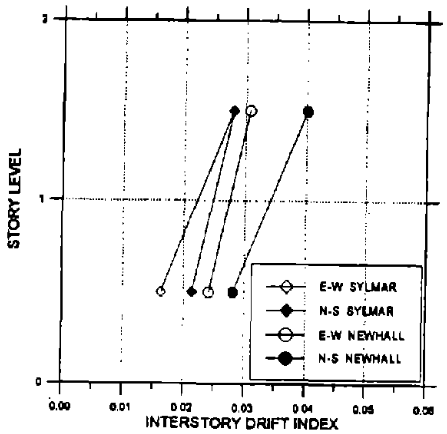

Fig. 13. DRIFT DEMANDS

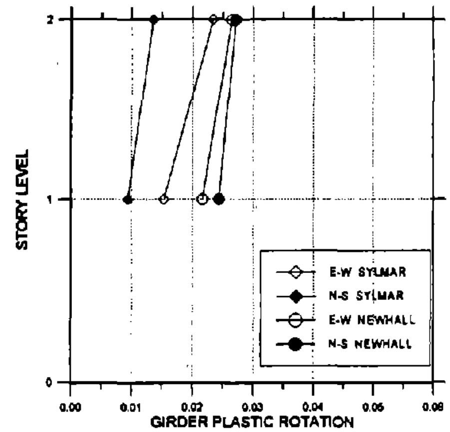

Fig. 14. ROTATION DEMANDS

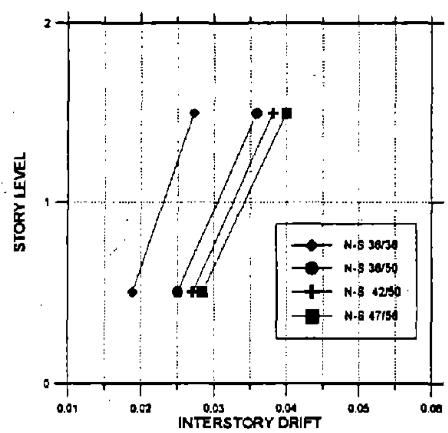

Fig. 15. YIELD EFFECT (N-S, DRIFT)

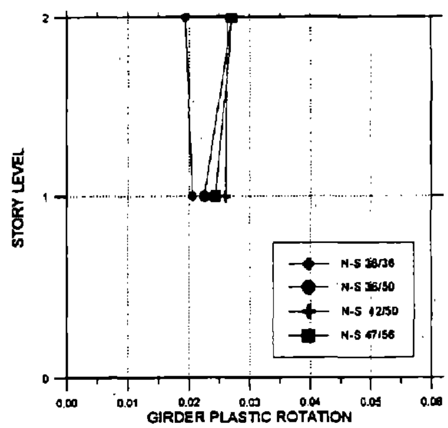

Fig. 16. YIELD EFFECT (N-S, ROTATION)

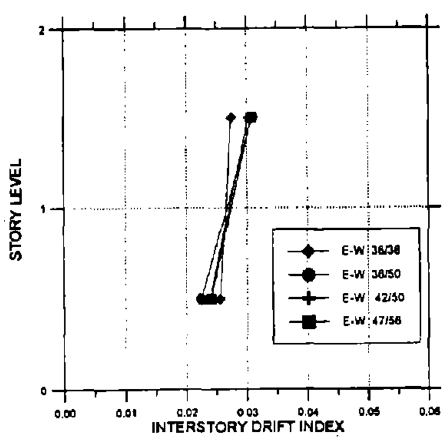

Fig. 17. YIELD EFFECT (E-W, DRIFT)

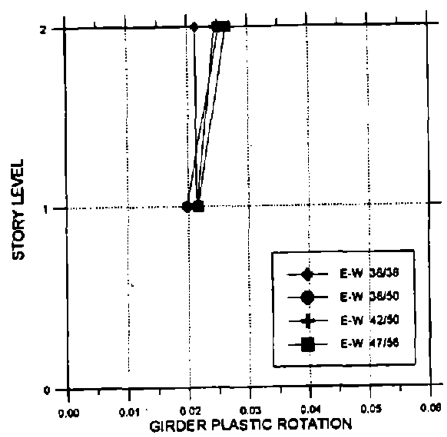

Fig. 18. YIELD EFFECT (E-W, ROTATION)

Redundancy and Ductility in Steel Moment Frames

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SUMMARY

Steel moment frames have been used for many years. Prior to the January 17, 1994, Northridge Earthquake, these structures were regarded as the premier structural system for seismic design. During that earthquake several hundred steel frame buildings sustained a wide range of cracking damage. None of these steel frame buildings collapsed, and there was no loss of life. However, the economic cost of this damage is large, and the future performance of these structures is unclear. As a result, the reputation of this structural system has been severely damaged.

This paper will summarize the history of the development of steel moment frames. The use of steel moment frames started in the early 20th Century, and today's frames are very different from those early structures. The changes and the motivation for these changes will be discussed. The seismic behavior expected for steel frames of various vintages will be described. These changes were consistently motivated by the desire to enhance the economy and practicality of these steel frames, but it will be shown that the changes consistently resulted in reduced redundancy of the structural system.

The effect of the reduced redundancy in the structural system on the inelastic seismic performance of steel frames will be discussed. It will be shown that redundancy of the structural system consistently resulted in good seismic performance despite deficiencies in performance. Virtually all of this redundancy has been traded away in our present day steel moment frames, and it will be shown that the lack of redundancy is a major contributor to the damage noted during the Northridge earthquake.

EARLY HISTORY OF STEEL MOMENT FRAMES

Steel moment frames have been used since very early in the 20th Century. Prior to the 1920's, these frames were constructed as complex built up members with gusset plate and built up connections as illustrated in **Figure 1**. The built up members were employed because labor costs were low, and the built up design allowed versatility in that a wide range of members could be constructed with a small number of sizes and shapes. This permitted shipping of large quantities of a few steel sizes, and avoided shipping delays when last minute changes were required at the job site. These members and connections were riveted, and the entire steel frame was encased in concrete. Further, few if any of these steel structures were designed for seismic loading, since only wind load was considered prior to about 1930. These buildings invariably included many stiff, strong unreinforced masonry walls and partitions. Structural engineers relied upon these walls and partitions to help resist wind and earthquake loads, since they did not rely on extensive calculations, but employed observations of the past performance of these buildings in the design.

Changes in steel frames began to evolve around 1920. Labor costs for the built-up elements were increasing. AISC Specifications (1928) were first developed in this period, and as a result standard riveted connections such as those illustrated in **Figure 2** with standard hot rolled shapes for beams and columns became the normal practice. These connections employed riveted angles and T-sections, and the member and the connection were encased in concrete for fire protection. Unreinforced masonry curtain walls and partitions were still used, and the combined effect of the added strength and stiffness provided by these walls and the composite action due to the encasement provided a major portion of the structural strength and stiffness. Seismic design forces were considered in these structures, but the seismic design forces were simplified and often smaller than those used today. These early structures were highly redundant in that every beam-column connection was a moment resistant connection, and a large but uncalculated stiffness and resistance was provided by nonstructural elements.

FIGURE 1.
BUILT-UP MEMBERS USED IN EARLY 1900'S

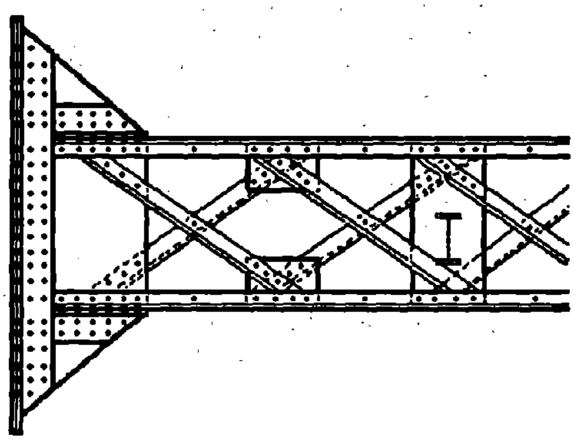
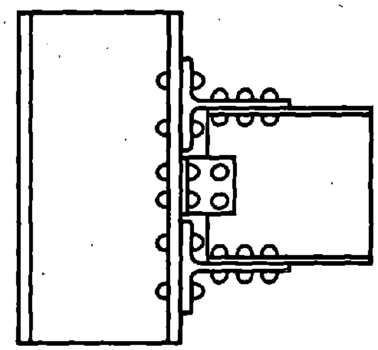


FIGURE 2.
TYPICAL RIVETED CONNECTION



The connections and construction described above and illustrated in **Figure 2** were used until the mid-1950's or early 1960's. At that time, high strength bolts replaced the rivets, although connection details such as those illustrated in **Figure 2** were still employed. Concrete encasement was also discontinued in favor of other lighter fire protection materials. By this time, the seismic design procedures had evolved to a period and mass dominated procedure, and therefore engineers began to reduce the mass and stiffness of the structure, since these reduced the design forces. However, buildings of this era still had a substantial uncalculated strength and stiffness due to nonstructural elements, and they were very redundant since moment resisting connections were used at every beam column joint. This construction continued into the early 1970's.

Engineers commonly note that no lives have been lost in these early steel structures during past US earthquakes, and none of these buildings have collapsed. As a consequence, engineers often assume that the inelastic performance of these early structures must be very good. In fact, this is often not the case. The hysteretic behavior produced by these early connections is invariably pinched and deteriorating. Very little energy is dissipated. The capacity for inelastic rotation in these older connections was often large as illustrated in the moment-rotation hysteresis curve illustrated in **Figure 3**, but this rotational capacity was highly dependent upon failure mode. **Figure 4** illustrates a moment rotation curve with limited rotational capacity expected for a connection with an undesirable failure mode. Engineers did not calculate the strength and failure modes of these early connections, and so the failure mode that should occur is dependent on chance. Thus, the good performance of these older buildings is not provided by superior steel frames, but it is provided by redundancy. The redundancy meant that the large strength and stiffness provided by nonstructural elements prevented the connections from achieving the deformations needed to result in failure. Further, the large number of moment resisting connections provided redundancy, which meant that distress exhibited by a few isolated connections during a major earthquake had no detrimental impact on the overall structural performance.

FIGURE 3.
MOMENT ROTATION OF RIVETED CONNECTION WITH LARGE ROTATIONAL CAPACITY

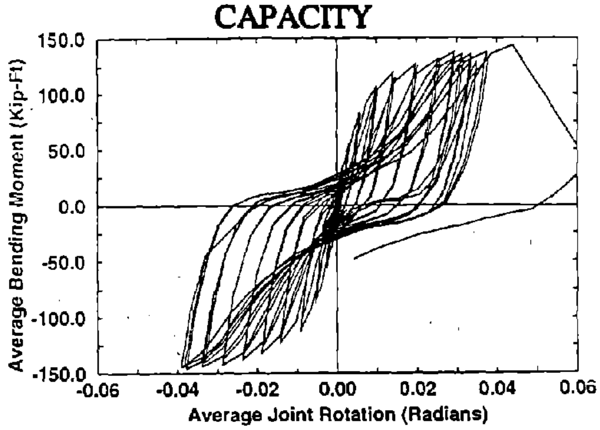
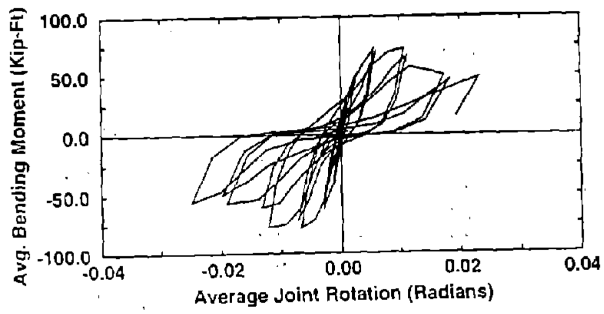


FIGURE 4.
MOMENT ROTATION OF CONNECTION WITH SMALL ROTATIONAL CAPACITY



LATER HISTORY OF STEEL MOMENT FRAMES

In the late 1960's and early 1970's, the seismic practice for steel moment frames evolved to the fully restrained (FR) bolted web-welded flange moment resisting connection illustrated in **Figure 5**. This connection was chosen because of extensive research by Popov and Pinkney (1968, 1969), Popov and Stephen (1970), Krawinkler etal (1971), and Bertero etal (1973). These tests showed that better inelastic cyclic behavior was achieved with the fully welded flanges and bolted webs as illustrated in **Figure 6** than with bolted connections such as used in earlier structures. The hysteresis were full, and the strength and stiffness remained stable through large inelastic deformations. Further, this connection developed the full plastic capacity of the beam rather than developing a rather brittle failure in the connection or net section. It must be noted that the experiments used to justify this FR connection were on beams with a depth no greater than 24 inches, however this was not a serious limitation since steel frames of the early 1970's seldom had beams greater than this depth. Therefore, rigid, fully restrained (FR) connections such as those schematically illustrated in **Figure 5** became the normal connection for seismic design. These FR connections have been used for seismic design for more than 20 years. They have a full penetration weld connecting the beam flange to the column, and an erection plate bolted to the web for transfer of shear force. Stiffeners or continuity plates are often required to prevent local damage to the connection, and panel zone stiffeners or doubler plates may be required to control panel zone yield and deformation.

While this early research established the general directions of seismic design, a number of changes in the design specifications and professional practice occurred during the years that followed. In 1988, the Uniform Building Code (1988) changed to increase the shear strength of panel zones. This increase was based on observations of the excellent ductility provided by panel zone yielding in tests by Bertero (1973), Krawinkler (1971), Popov (1986), and others. These tests showed that panel zone yielding results in reliable energy dissipation with considerable strain hardening. Building codes increased the rated shear strength of the panel zone in recognition of this added resistance due to strain hardening. The increased panel zone strength rating meant that steel frames built since 1988 will sustain larger inelastic deformation in the panel zone during an earthquake, since they initially yield at a smaller seismic event. Another change to the Uniform Building Code (1988) required supplemental welding of the beam web to the shear plate, because of a test program by Tsai and Popov (1988). A later study by Englehart and Husain (1993) examined the behavior of FR moment frame connections with W18, W21, and W24 beams, and these tests showed a disturbing lack of ductility in some specimens.

FIGURE 5
FR BOLT WEB-WELDED FLANGE CONNECTION

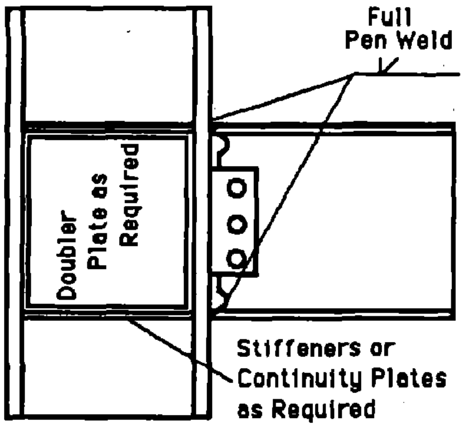
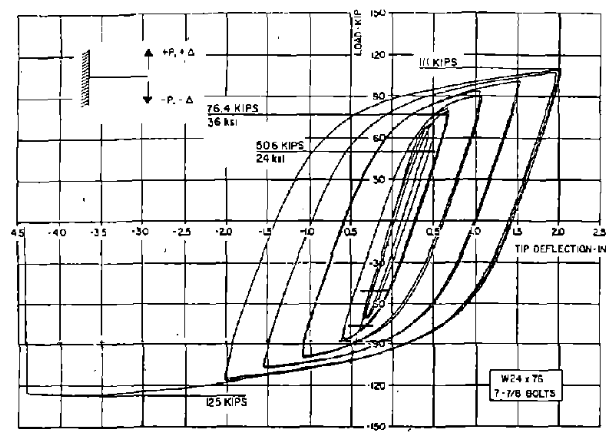


FIGURE 6
MOMENT ROTATION BEHAVIOR FROM EARLY FR CONNECTION TEST



The Northridge Earthquake occurred on January 17, 1994, and many steel frames experienced cracking during the earthquake. The cracking had a number of different variations, and many of the variations had not been observed in past experiments. A number of tests performed under the SAC Program (1994, 1995) at the University of Texas at Austin, the University of California at Berkeley and the University of California at San Diego have been performed since this earthquake.

REDUCED REDUNDANCY RESULTING FROM THESE CHANGES

These changes resulted in a dramatic reduction in redundancy in steel frame buildings. Until the late 1970's, FR connections were used at all beam-column connections in the structural system. This resulted in good distribution of lateral stiffness and resistance, and member and connections were relatively small. However, the FR connections are relatively costly, and engineers began to minimize their use. At first, perimeter frames were used to replace frames with FR connections at all beam-column connections, since perimeter frames resulted in similar translational and torsional stiffness while significantly reducing the number of FR connections. The total seismic resistance of the structure does not decrease when this concentration of seismic resistance is employed. Therefore, increased bending moments and stiffness must be developed within individual members and connections. Perimeter frames resulted in larger members and connections. However, even more dramatic increases in member, flange and weld sizes were produced by engineers, who concentrated the seismic resistance into individual isolated frames or bays of frames. This later change significantly reduced the redundancy of the structural system, since individual members and connections played a far greater role in resisting seismic loads. Further reductions in redundancy were contributed by the reduced stiffness and resistance provided by nonstructural

$$FDR = \frac{\Delta_{maximum}}{\Delta_{elastic}} . \qquad \text{(Equation 1)}$$

The elastic displacement, $\Delta_{elastic}$, and the maximum displacement, $\Delta_{maximum}$, were the deflection of an equivalent cantilever beam with its elastic stiffness at the development of the nominal plastic load capacity and the maximum deflection prior to fracture of the specimen, respectively. The past data shows that the ductility ratios for FR bolted web-welded flange connections vary from less than 1.0 to nearly 16. There were large scatter of test results, and wide variations in the specimens tested. However, beam depth has a strong correlation with specimen ductility as illustrated in **Figure 7**.

FIGURE 7
DUCTILITY RATIO AS A FUNCTION
OF BEAM DEPTH

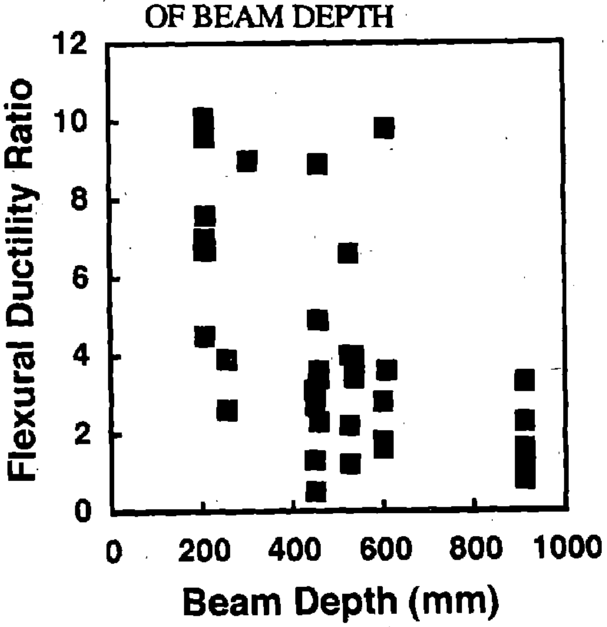
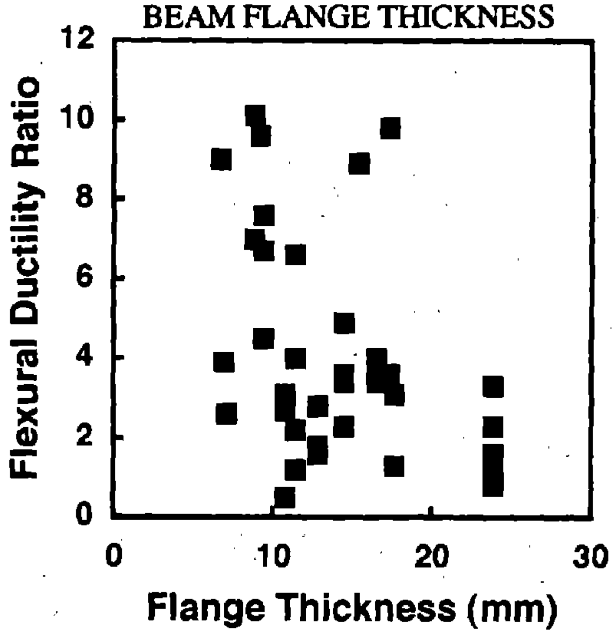


FIGURE 8
DUCTILITY RATIO AS A FUNCTION
BEAM FLANGE THICKNESS



Specimens with supplemental reinforcement such web welding, additional flange plates, and other connection reinforcement often obtained significantly larger ductility and are excluded from the figure. Specimens with weak axis bending of the column or specimens with significant panel zone yield have somewhat smaller ductility than that shown in **Figure 7** and are also excluded. Panel zone yielding produces very good overall hysteretic behavior, but panel zone yielding reduces the ductility achievable in the beams themselves. This is significant since relatively large amount of strain hardening occurs with panel zone yielding.

Thick beam flanges also reduce flexural ductility as illustrated in **Figure 8** Thick flanges require larger full penetration welds at the column flange, and this may contribute more potential flaws, and this may also lead to early fracture. There clearly are other factors which affect the connection ductility. For example, changes in the yield stress of the steel and the use of relatively non ductile E70T-4 weld electrode clearly contributed to the Northridge experience. However, member sizes resulting from reduced redundancy represent a major source of potential cracking during the Northridge earthquake.

walls and partitions. This reduction was small in 1970, when FR connections were used at all beam-column connections, but walls and partitions became lighter and more flexible in the years that followed. During the late 1980's, many office buildings were constructed without any full height partitions. This reduction in the uncalculated strength and stiffness became particularly significant because some engineers used the reduced frame stiffness to increase the computed period of the building and to further reduce the seismic forces used to establish the strength limits and drift control.

The magnitude of this effect can be illustrated by comparing the designs of four steel frame buildings built in California during these different periods. Building A is a 26 story steel frame building constructed in downtown San Francisco in the mid 1920's with riveted connections such as those illustrated in Figure 1. Typical column spacing for this building was in the order of 17 feet, and typical beams for these spans were no more than 22 in. (550mm) deep and beam weight of approximately 65 lb/ft. The very longest column spacing in this building were approximately 30 ft. (9 m) with beam depths 26 to 30 in. (650 to 750 mm) and beam weights in order of 100 lb/ft. Building B was built in San Francisco in the mid 1960's. This building is 22 stories and has similar connection details to those used in Building A except that high strength bolts were employed. The typical column spacing was approximately 27 ft. (approx. 8 m.) and the heaviest beams are W27's with weights less than 100 lb/ft. Building C was built in mid-1970's and is 30 stories tall with FR connections at all beam-column joints. The beam spans are approximately 30 ft. (9 m.). This is the tallest building of the four and it has the largest column spacing, and so the beam sizes are expected to be somewhat larger that those used in Buildings A and B. Beam depths vary between W24's and W36's with W33 being typical and with beam weights being 110 to 150 lb/ft. The very heaviest beams in the bottom stories of this 30 story structure are W36x260. These heaviest sections are somewhat an anomaly, since are they only used on the first story where the story height is taller than that used in any of the other buildings. Building D is a 17 story building located in the San Fernando Valley, and is described in the analytical studies [Paret and Sasaki (1995)] completed as part of the SAC Phase I program. The building was built in the mid 1980's, and 2 bays of seismic framing are located each of the four perimeter walls. This building also has column spacing similar to that used in Band C, but it is the shortest and lightest building of the four buildings. Beam sizes in the moment frame are W36x300 for the bottom framing and even the top story requires a W36x150 section. The building is shorter and lighter than the two earlier examples, and so its seismic design forces should be smaller than these older buildings, but the reduced redundancy resulted in significant increases in the depth and weight of the beams in the moment frames.

EFFECT OF CHANGE ON SEISMIC PERFORMANCE

The prior discussion has shown that there has been a steady decrease in the redundancy of steel moment frame buildings in the past 20 years. This reduced redundancy can be seen in the dramatic reduction in the number of FR moment resisting connections in a given structure and in the significant reduction in uncalculated reserve strength provided by nonstructural elements. The consequence of this reduction is a significant increase in the depth and weight of beams used in steel moment frames. These heavier sections require larger welds and the structural performance is more dependent on each of these welds. It is instructive to examine the results of past experiments to understand the effect of these changes on the expected seismic performance. The data obtained from more than 120 experiments on steel moment frame connections and subassemblages was analyzed and compared by Roeder and Foutch (1996). For this evaluation, the flexural ductility ratio, FDR, was defined where

CONSEQUENCE OF REDUCED REDUNDANCY IN NORTHRIDGE

Present estimates indicate that cracking has been noted in several hundred steel frame buildings after the Northridge Earthquake. A building damage database developed by Bonowitz and Youssef (1995) provide useful information regarding the extent and type of cracking. Type W1 cracking is excluded from the analysis that follows, since this cracking was determined only by nondestructive inspection methods, and may be caused by poor construction quality control as opposed to earthquake damage. The database shows that cracking was more common in newer buildings. Approximately 32% of the frames inspected had cracking in the weld, beam or column, and 15.7% of the inspected frames had cracking in the beam or column. However, approximately 50% of the inspected frames designed after 1990 had cracking in the welds, beams, or columns, and 27.7% had cracking in the beams or columns. Buildings designed before 1980 had cracking in beams, columns, and welds in approximately 24.5% of the frames inspected, and approximately 12.5% had cracking in beams and columns. This comparison indicates that older structures had less tendency toward cracking than average, while the newer steel frame buildings had a greater tendency toward serious cracking. In fact, the analogy may be stronger than suggested by this statistic, since most of the cracking observed in buildings designed prior to 1975 are concentrated in a single building. If this building is deleted from the data, buildings designed before 1975 had cracking in the welds only approximately 3% of the frames inspected, and none of the frames had cracking in the beams and columns. Statistics of this type must be used with some care, however the data suggests that recent changes in the practice may have contributed to the cracking. The reduced redundancy is one of these major changes.

The database also shows that the cracking damage was more significant in steel frames with deep beams and thick beam and column flanges. An average of approximately 15.7% of the frames inspected as part of the survey had cracking in the beams or columns. However, none of the frames with beam depths less than 20 inches had cracking of this type, and approximately 18.5% of the frames with beams 30 inches or greater had these types of cracking. The data for the W21, W24, and W27 beams are intermediate, but the issue is muddied for these intermediate depths because many (possibly most) of these intermediate depth beams with cracking were heavy W24 sections with thick flanges. The concentration of damage in deeper and heavier beams is an important observation, since deeper beams are a natural consequence of the reduced redundancy used in recent years.

SUMMARY AND CONCLUSIONS

This work has shown that there has been a significant reduction in redundancy in steel frame buildings in the past 20 years. The reduced redundancy has resulted in larger member sizes in these newer frames. At the same time experimental results show that these larger member sizes provide much smaller ductility than the more modest sized members used in older buildings. Examination of the Northridge data base provides further corroboration of these observations in that deep beams and newer structures have had a significantly larger concentration of seismic damage than older structures with shallower members. It is clear that there are other factors which contributed to the Northridge damage to steel frames, but this analysis indicates that a return to increased redundancy in steel frames would be a major step in the right direction.

ACKNOWLEDGEMENTS

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Cyclic Performance of Beam Top Flanges in Steel Moment Frame Connections

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SUMMARY

Damage survey of steel moment frame connections conducted after the 1994 Northridge earthquake revealed widespread damage at the beam bottom flange level. This finding, despite incomplete information of beam top flange groove welded joints, significantly influences not only the research direction but also the design practice for seismic repair, retrofit, and new construction. Based on a limited number of test results, it was found that the quality of top flange groove welded joints is not much better than that of the bottom flanges. For seismic repair and retrofit, concrete slab cannot be relied on to reduce the vulnerability of top flange fracture of pre-Northridge moment connections. The existence of a backup bar that creates a notch condition appears to be very detrimental.

INTRODUCTION

The January 17, 1994 Northridge earthquake caused extensive damage to moment connections in steel moment frame buildings. The failures were predominantly brittle fractures in or around the beam bottom flanges to column flange groove weld. Based on the available damage survey (Youssef et al. 1995), most of damage occurred at the bottom flange level. Nevertheless, such a database may be skewed for several reasons. Bonowitz et al. (1995) reported that "Weld cracks were reported at the beam bottom flange about three times as often as at the beam top flange. Base metal fractures at the top of the connection were extremely rare." However, they also cautioned the reader that "Top inspection was substantially incomplete compared to bottom inspection.... Because access to beam top flanges is frequently obstructed by slabs and perimeter walls, it is reasonable to expect that lower damage rates at the top of the connection are due in part to limited post-earthquake inspection and testing." Under such circumstances, the findings of predominant beam bottom flange fracture after the Northridge earthquake might have misled engineers to focus the inspection and strengthening effort more on the bottom flanges and overlook the potential vulnerability of top flanges.

Based on the current thinking that the Northridge earthquake is predominantly a "bottom-flange" earthquake, several reasons have been given to explain this phenomenon (SAC 1995). First, it is generally believed that the filler metal (E70T-4) used for making the complete joint penetration groove weld does not have sufficient notch toughness for seismic applications (Xue et al. 1996). The use of backup bar and leaving it in place after welding creates a notch condition (see Fig. 1). Second, the presence of beam web not only interferes with the making of a continuous groove weld across the flange width but also makes it difficult for ultrasonic inspection around the cope hole, where welds overlap and the stress may be the highest due to stress concentration (Popov 1986). Unfortunately, such a notched condition coincides with the location of extreme fiber of the beam under positive bending. These two reasons imply that the quality of top flange welded joints is significantly better than the bottom one. Third, it has been thought that the presence of a concrete slab causes the neutral axis of the steel beam to shift upward in positive bending (Leon et al. 1996), making the bottom flange even more critical than the top flange.

Despite the lack of sufficient information on top flange fractures, the engineering community has been led to believe that the welded joint of the top flanges is more forgiving and is much less a concern for earthquake resistance. After the Northridge earthquake, such a belief has been reflected not only in steel research activities, where heavy emphases have been placed on improving the beam bottom flange for the repair and retrofit of existing steel buildings, but also in new construction, where less stringent requirements (e.g., leaving the top flange backup bar in place) have been adopted from time to time by design engineers.

Is it possible that the difference in welding quality between the top and bottom flanges not as significant as we thought? Is it possible that our current thinking and effort to improving the seismic performance of steel moment connections may create a large difference in quality between the two flanges? Furthermore, is it likely that the next major seismic event will turn out to be a "top flange earthquake?"

OBJECTIVE AND SCOPE

The objective of the paper attempts to examine the following postulation:

> "The qualities between the top and bottom flange groove welded joints of the pre-Northridge type moment connections do not differ appreciably. Yet the modest difference in weld quality between two flanges is enough the cause the bottom flange to fracture earlier; this, to some extent, protects the top flange from further fracturing."

Five identical bare steel specimens that were tested either statically or dynamically at UCSD were used to evaluate the relative cyclic performance of both flanges. Three of them that were repaired with haunches but with different treatments of the beam top flanges were also included in this evaluation. To assess the effect of the concrete slab on beam top flanges, two retrofitted specimens, one with and one without composite slab, were considered in the comparison. The effect of backup bars on the cyclic performance was examined.

PRE-NORTHRIDGE STEEL MOMENT CONNECTIONS

A total of five nominally identical specimens with pre-Northridge type of design and construction have been tested (see Fig. 2). Among all the pre-Northridge test specimens, only Specimen 2 had the backup bar of the bottom flange removed for weld repair. The first three specimens were tested for SAC using the conventional quasi-static testing procedure (Uang and Bondad 1996a), while the last two specimens were tested dynamically for an NSF-funded project (Uang and Bondad 1996b). Sample response of one statically loaded specimen and one dynamically loaded specimen is presented in Fig. 3, and the fracture mode is summarized in Table 1.

From Table 1, it is obvious that the quality of the top flange groove welded joints is by no means much better than that of the bottom flanges. Figure 4 compares the plastic rotation and energy dissipation capacities of all five specimens. It is observed that dynamic loading tends to produce inferior cyclic performance of the pre-Northridge moment connections.

REPAIRED STEEL MOMENT CONNECTIONS

Both Specimens 1 and 4 experienced bottom flange fracture and were subsequently repaired with the addition of a triangular haunch beneath the beam. Prior to repair, ultrasonic testing of top flange welded joints of both damaged specimens did not reveal rejectable weld defects. Hence, no improvement was made to the top flange of the repaired Specimen 1. For the repaired Specimen 4, however, it was decided to add two vertical rib plates beneath the top flange in order to strengthen the existing welded joint. In both cases the top flange backup bars were not removed.

Test results showed that while the repaired Specimen 1 was able to develop a plastic rotation of more than 0.02 radian before the top flange fractured near the welded joint under static loading, the top flange welded joint of the repaired Specimen 4 ruptured under dynamic loading, leaving only the vertical ribs to transfer the top flange force to the column. Although repaired Specimen 4 performed well, it did shows the vulnerability of the pre-Northridge style top flange welded joint.

Specimen 5 was also repaired by adding a haunch beneath the beam. The fractured top flange was welded to the column by using a better filler metal (E71T-8) with the backup bar removed after welding. The repaired specimen performed very well under dynamic loading. It was able to dissipate significant amount of energy without fracturing the top flange groove welded joint.

Testing of this repaired specimen clearly shows the beneficial effect of removing the top flange backup bar to eliminate the notch condition, although the use of filler metal with better notch toughness is also helpful.

RETROFITTED STEEL MOMENT CONNECTIONS—SLAB EFFECT

Two identical pre-Northridge test specimens, one with and the other one without concrete slab, have been retrofitted and tested as a part of the NIST-funded project to study the retrofit scheme of introducing a reduced beam section, or "dogbone", to the bottom flange only (Fig. 5). (With the presence of concrete slab, cutting a portion of the top flange is not an easy task.) Each specimen consisted of a W14X426 column (A572 Gr. 50 steel) with W36X150 beams (A36 steel). The composite specimen incorporated an 8-ft wide lightweight concrete floor slab. Following the typical California construction practice, the slab consisted of 3-1/4 in. concrete over 3 in. deep metal deck. Headed shear studs (5/8-in diameter) were placed every 12 in along the beam length. To reflect the commonly accepted belief that bottom flange is much more vulnerable to brittle fracture, the backup bar of the bottom flange was removed. It was also decided not to improve the top flange welded joint in order to achieve a more economic retrofit.

Quasi-static testing was conducted on both specimens. The load versus beam tip deflection relationships of both specimens are shown in Fig. 6. Both beams of the bare steel specimen, NIST-1, experienced brittle fracture in the top flange welded joints, delivering a plastic rotation of about 0.01 radian. The cyclic performance of the composite specimen, NIST-1C, is very similar to that of the bare steel specimen. Brittle fracture of one beam top flange welded joint occurred at about the same loading step as the bare steel specimen (see Fig. 6). Cracks in concrete slab developed at about half of the yield displacement amplitude. Therefore, the presence of concrete slab had little effect on the negative bending capacity of the beam and the connection behavior. Under positive bending, the slab was able to bear against the column. Measurements of strains in the concrete slab indicated that the slab effective width under positive bending is about the width of the column (see Fig. 6). Test results showed that the presence of concrete slab increased the positive bending capacity of the beam by about 10 to 20 percent.

CONCLUSIONS

The following conclusions can be made.

(1) The findings from available field survey that damage of steel moment frame connections occurred predominantly in the beam bottom flanges may be misleading and should be viewed with great caution.

(2) Based on the limited number of cyclic tests of pre-Northridge type of steel moment connections at UCSD, it was found that, for the particular beam size (W33x99) tested, the performance of top flange groove welded joints are not much better than the bottom ones. The cyclic performance is even poorer under the dynamic loading condition.

(3) For repair or retrofit, enhancing only bottom flanges tends to create a even greater discrepancy in quality between top and bottom flange groove welded joints. Under such circumstances, top flange welded joints with the presence of backup bars are very vulnerable to brittle fracture, making future repair even more difficult.

(4) Concrete slab cannot be counted on to assist beam top flange for seismic retrofit or repair. For new construction, the practice of leaving the backup bar of top flange in place should also be studied carefully even for the case that notch-tough filler metal is used.

ACKNOWLEDGMENTS

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Specimen	Observed Fracture Mode
UCSD-1	beam bottom flange (between groove weld and column face)
UCSD-2	beam top flange (between groove weld and beam flange)
UCSD-3	beam top flange (in groove weld)
UCSD-4	beam bottom flange (fracture between groove weld and column face propagated into column flange)
UCSD-5	beam top and bottom flanges between groove weld and column face (fracture on the beam bottom flange propagated into column flange)

Table 1 Observed Failure Mode (SAC and NSF Specimens)

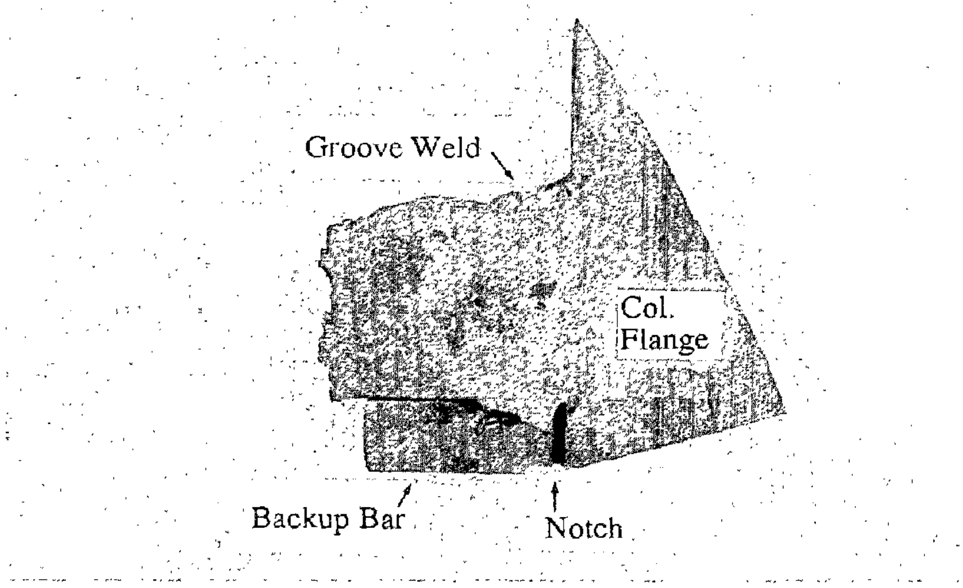
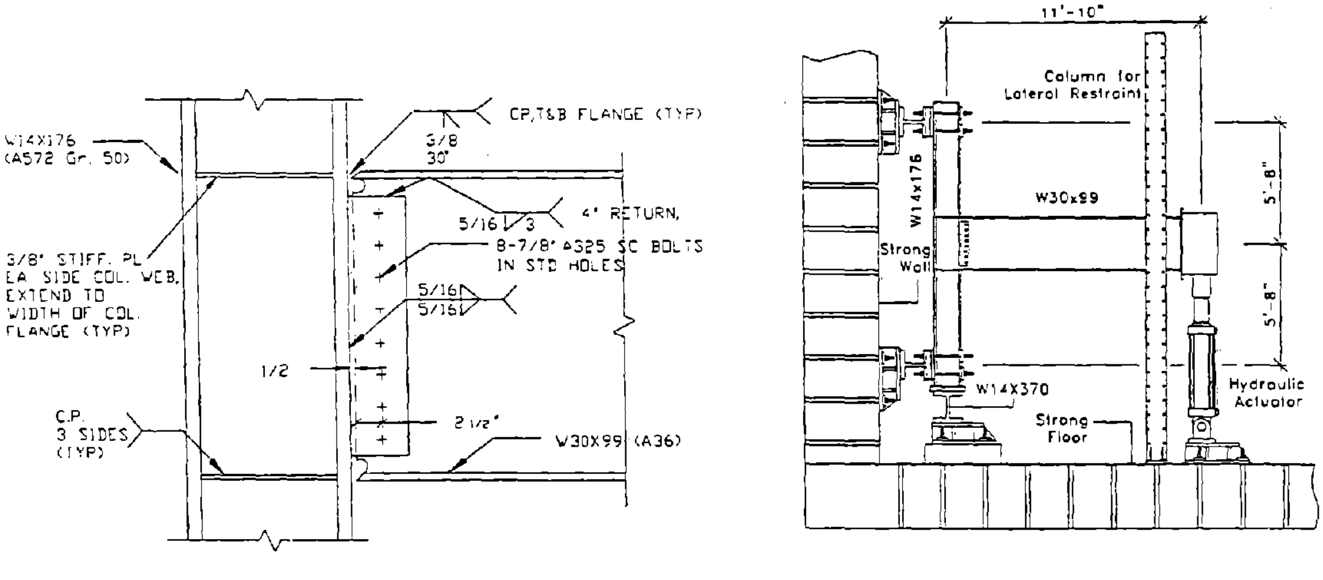


Fig. 1 Notch Condition Created by Backup Bar
(Courtesy of J. Patridge)



(a) pre-Northridge Moment Connection Details

(b) Test Setup

Fig. 2 Test Setup and Moment Connection Details
(SAC and NSF Specimens)

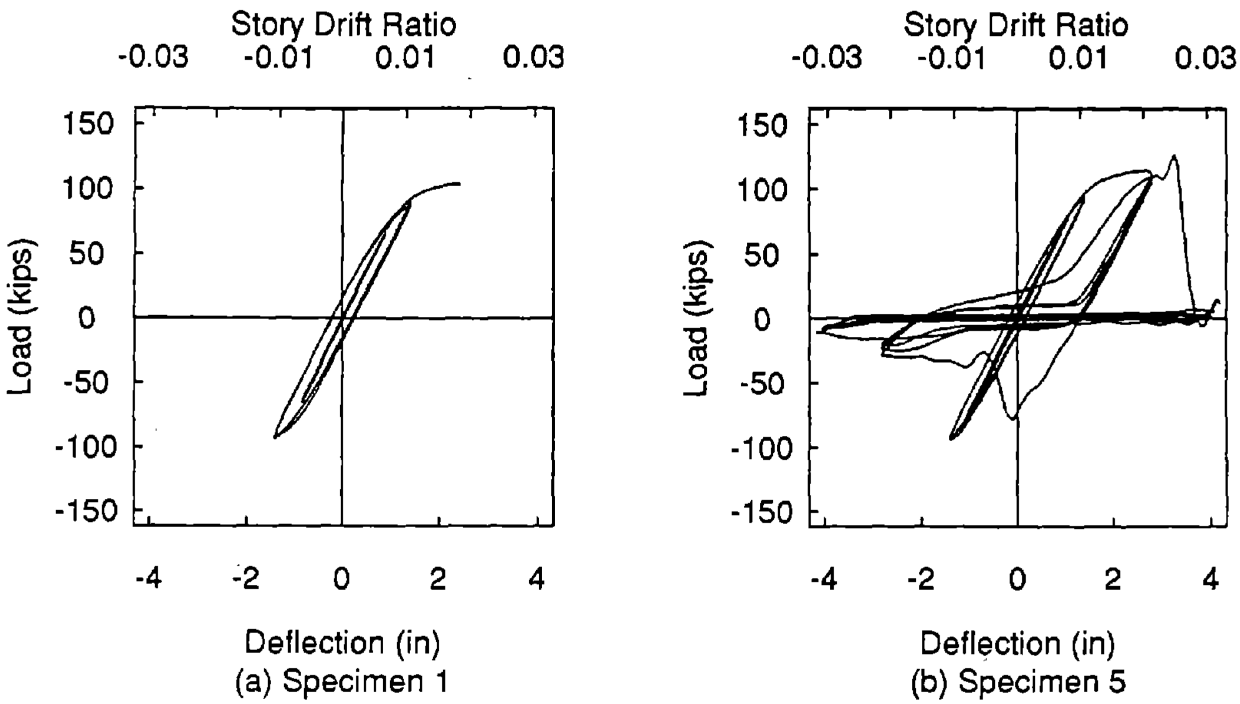
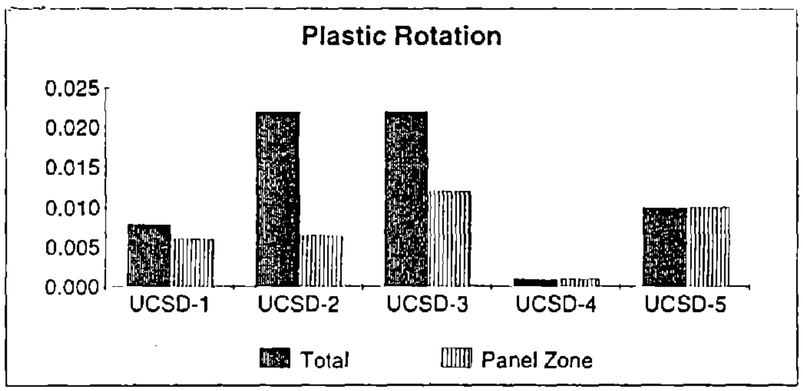
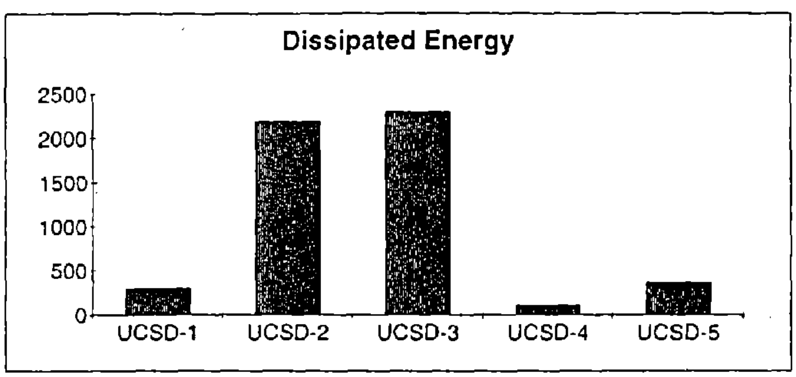


Fig. 3 Load versus Beam Tip Deflection Relationships
(SAC and NSF Specimens)

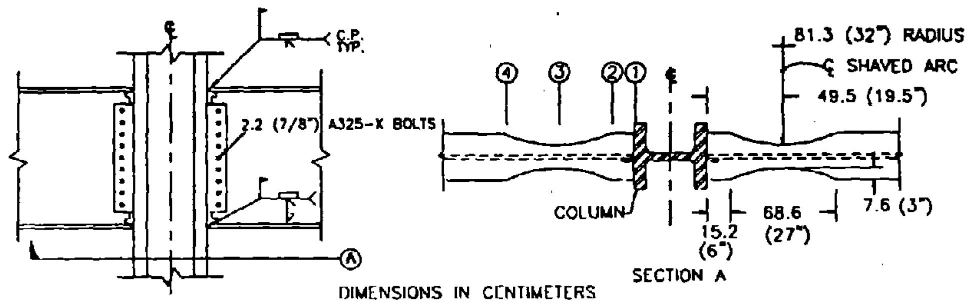


(a) Plastic Rotation

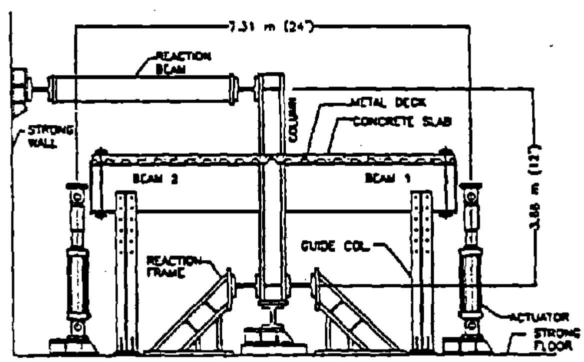


(b) Hysteretic Energy Dissipation

Fig. 4 Comparison of Plastic Rotation and Energy Dissipation Capacities
(SAC and NSF Specimens)



(a) Retrofitted Moment Connection Details



(b) Test Setup

Fig. 5 Test Setup and Moment Connection Details (NIST Specimens)

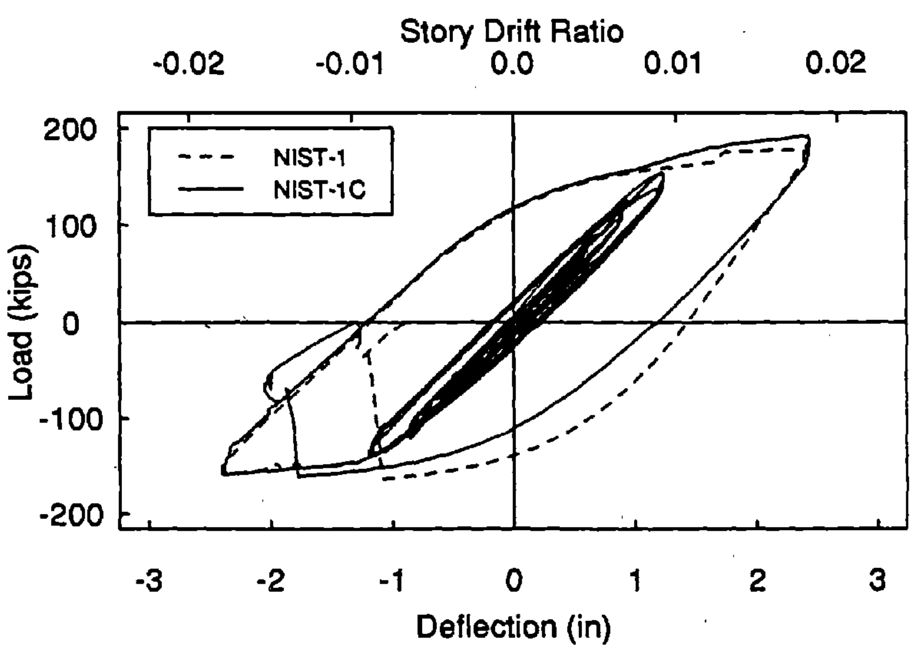


Fig. 6 Load versus Beam 2 Tip Deflection Relationships (NIST Specimens)

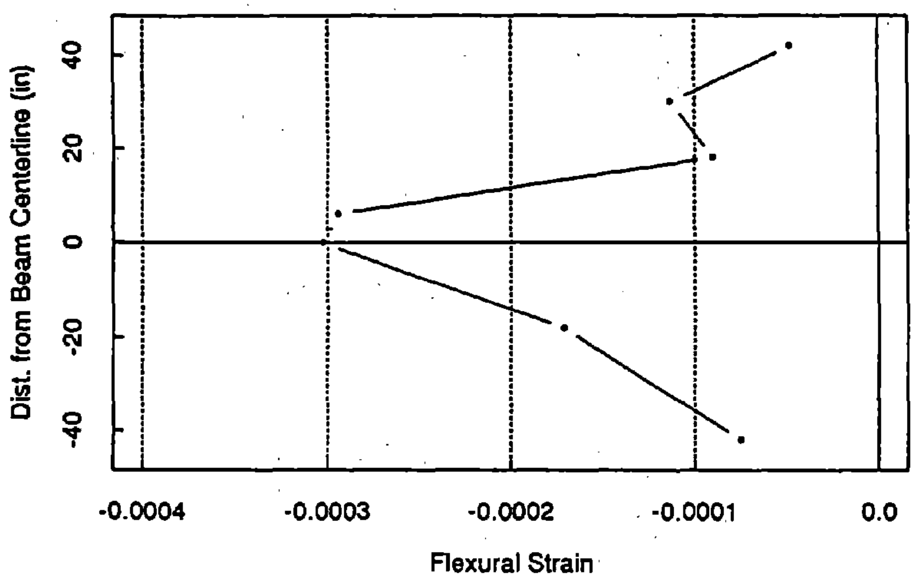


Fig. 7 Concrete Strain Profile across Beam 1 Slab (Specimen NIST-1C)

A Framework for Performance-Based Earthquake Resistive Design

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SUMMARY

SEAOC's *Vision 2000* project and BSSC's *NEHRP Guidelines for Seismic Rehabilitation of Buildings* have made fundamental contributions to the development of performance-based engineering by introducing the concepts of design performance objectives, acceptance criteria tied to performance level, and the use of alternative analytical techniques for performance evaluation. The proposed *1997 NEHRP Provisions for Seismic Regulation of Buildings and Other Structures* also make an important contribution, by attempting for the first time to define the margin of safety inherent in buildings conforming to these provisions, and in the sense of a Load and Resistance Factor Design procedure, by directly incorporating this presumed margin in the definition of the loading function.

Key areas of development, required to provide true performance-based capability in future design provisions include the incorporation of a specific serviceability level performance evaluation procedure, verification of the reliability actually inherent in buildings of different structural systems conforming to the provisions and the development and refinement of new analytical evaluation procedures capable of predicting building performance with reduced uncertainty.

INTRODUCTION

The international earthquake engineering community has mobilized in an effort to develop methods of performance-based earthquake engineering. As defined by the Structural Engineers Association of California (SEAOC), in their *Vision 2000* report (SEAOC, 1988, p.1-C), the intent of performance-based earthquake engineering is to provide methods for siting, designing, constructing and maintaining buildings, such that they are capable of providing predictable performance when affected by earthquakes. As used here, performance is measured in terms of the amount of damage sustained by a building, when affected by earthquake ground motion, and the impacts of this damage on post-earthquake disposition of the building. The concept is not limited to buildings alone, but is generally applicable to all structures and their supported non-structural components and contents. While the framework proposed by SEAOC in *Vision 2000* appropriately addresses all aspects of the performance-based engineering including structural and nonstructural design, construction quality assurance and maintenance of building integrity throughout its life cycle, this paper focuses on the structural design aspects of the problem.

Inherently, the performance-based design concept implies the definition of multiple target performance (damage) levels which are expected to be achieved, or at least not exceeded, when the structure is subjected to earthquake ground motion of specified intensity. Though development of the principles of performance based design is in its infancy, guideline documents upon which our future building codes will be based are rapidly focusing and adopting performance-based approaches. Much of the early development effort has taken place in the preparation of the *NEHRP Guidelines for Seismic Rehabilitation of Buildings* (ATC, 1996), intended as a resource document for use in upgrading the performance of existing buildings. The principles initiated in that document, intended primarily for existing structures, were rapidly extended by SEAOC's Vision 2000 committee and suggested for application to design of new structures, and these same concepts, have now been proposed for adoption into the Commentary to the NEHRP Provisions (BSSC, 1997a.) where they will serve as the stated basis and design intent for earthquake engineering contained in the future *International Building Code*.

Though the name performance-based engineering is new, the basic concept of developing buildings and structures that will meet expected performance levels under different ground motion scenarios is certainly not. For more than 20 years, SEAOC has indicated that structures designed in accordance with its recommended lateral force requirements (SEAOC, 1996) would be able to meet a number of specific performance objectives, i.e. - resist minor earthquakes without damage; moderate earthquakes with limited structural and nonstructural damage; major earthquakes with significant damage to structural and non-structural elements, but with limited risk to life safety; and the most severe levels of earthquake ground motion ever likely to effect a site, without collapse. These same basic performance objectives, though more precisely and quantitatively defined, are being adopted by most performance-based engineering guidelines today. It is the quantitative nature of these objectives as adopted in recent efforts and the attempt at precision and reliability that sets contemporary efforts at performance-based engineering apart from earlier practice. In traditional practice, earthquake design has been explicitly performed for only a single design event level, at which a level of performance generally termed "life safety" has been targeted. While attainment of the other performance objectives cited by SEAOC is implied, no specific procedures are provided to allow evaluation of the ability of a structure to actually meet these objectives. Contemporary efforts at performance-based engineering are seeking to provide reliable methods of meeting these multiple performance goals through explicit design procedures.

PERFORMANCE BASIS

Nearly all engineering design is performance based. For loading other than earthquake, most structures have traditionally been designed for two performance levels - a serviceability level and a failure level. At service level loading, structures are designed to perform without damage and to maintain deflections below a level that would be troubling to occupants or supported systems. Structures are not specifically designed for failure level loads, however, they are proportioned such that under expected loading, the structure will provide an acceptable margin against the failure state. This basic approach is inherent in the strength design specifications, more recently termed Load and Resistance Factor Design (LRFD) approaches adopted over the last 25 years for all of the major material systems.

Although contemporary earthquake engineering procedures, patterned after ATC-3.06 (ATC, 1978), purport to be strength based, in the sense of being an LRFD approach, in reality they are not. In current earthquake engineering procedures, structures are provided with a minimum strength based on a fraction, (1/R), times the theoretical lateral strength demand that would be experienced were the structure to remain elastic. There has never been a serious attempt to define the margin against failure provided by this approach, for the various structural systems for which R values are specified. Instead, these R factors have been set based on judgment and in part, based on observation of structural performance in recent earthquakes, to provide the so-called life safety performance level for design level earthquake ground motions. This life safety level of performance has been defined only qualitatively in terms of poorly stated considerations of limiting damage to structural elements, maintaining egress for occupants, and preventing significant falling hazards.

TABLE 1
DEFINITIONS OF STRUCTURAL PERFORMANCE

Performance Level		Description
NEHRP Guidelines	Vision 2000	
Operational	Fully Functional	No significant damage has occurred to structural and non-structural components. Building is suitable for normal intended occupancy and use.
Immediate Occupancy	Operational	No significant damage has occurred to structure, which retains nearly all of its pre-earthquake strength and stiffness. Nonstructural components are secure and most would function, if utilities available. Building may be used for intended purpose, albeit in an impaired mode.
Life Safety	Life Safe	Significant damage to structural elements, with substantial reduction in stiffness, however, margin remains against collapse. Nonstructural elements are secured but may not function. Occupancy may be prevented until repairs can be instituted.
Collapse Prevention	Near Collapse	Substantial structural and nonstructural damage. Structural strength and stiffness substantially degraded. Little margin against collapse. Some falling debris hazards may have occurred.

Both SEAOC's *Vision 2000* (SEAOC, 1995) and the *NEHRP Guidelines* (ATC, 1996) have attempted to provide more quantitative definitions of building performance levels. Both have developed similar systems of designating building performance, though somewhat different terminology has been utilized. **Table 1,** below, summarizes the performance levels defined by these projects. The *NEHRP Guidelines*, in particular, have specified quantitative criteria, by which structural performance can be evaluated, relative to these levels. To accomplish this, the various components that comprise the structure are designated as either primary or secondary. Primary components are necessary to the lateral stability and resistance of the structure.

Secondary components are not, although they may be necessary to the vertical stability of the structure. In total, secondary elements can not comprise more than 25% of the total lateral force resisting stiffness of the structure, prior to the onset of damage. Consistent with true LRFD approaches, acceptance criteria for the Life Safety and Collapse Prevention performance levels are specified based on desired margins against failure, at the component level. **Table 2** summarizes the acceptance criteria for these two performance levels, for both primary and secondary elements.

TABLE 2
ACCEPTANCE CRITERIA FOR LIFE SAFETY AND COLLAPSE PREVENTION PERFORMANCE LEVELS[1]

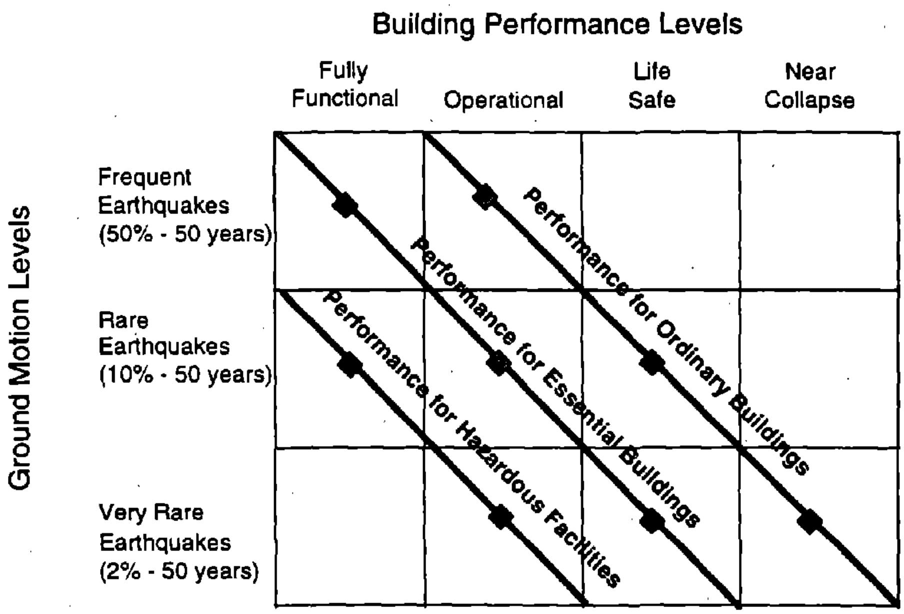
Performance Level	Primary Component	Secondary Component
Life Safety	75% of the deformation at which significant loss of lateral force resisting strength occurs	100% of the deformation at which significant loss of lateral force resisting strength occurs
Collapse Prevention	75% of the deformation at which loss of vertical load carrying capacity occurs, but not more than the deformation at which significant loss of lateral force resisting strength occurs	100% of the deformation at which loss of vertical load carrying capacity occurs

1. The acceptance criteria indicated apply to buildings for which nonlinear analytical methods are used to predict component demands. An additional reduction factor, of 0.75, is applied against these acceptance criteria when linear methods of analysis are used to predict component demands.

The *Vision 2000* document recommends that buildings be constructed, based on their intended occupancies and uses, to meet the performance objectives indicated in **Figure 1**. In the figure, each combination of an earthquake return period and performance level, indicated by a "◆," represents a specific design performance objective. The intent is that ordinary buildings provide a low risk that life be endangered as a result of the performance of the building in any earthquake likely to effect it; and that for frequent earthquakes, the building user not be burdened with extensive repairs or loss of use; that buildings required for emergency response and essential public function have a low risk of being damaged beyond a level that would permit their use, and; that facilities housing systems and materials that would pose a hazard to many persons if released, have a low risk of damage resulting in such release. The *NEHRP Guidelines* suggest similar performance objectives as the basis for rehabilitation design for existing structures and specifically recommend that a performance evaluation be performed for each specifically intended performance objective. The performance evaluation consists of a structural analysis with computed demands on structural elements compared against specific acceptance criteria provided for each of the various performance levels. This is in contrast to the approach taken by current building code provisions, wherein a single performance evaluation is required, for the

Life Safety performance level at a specified level of ground motion, termed the Design Basis Earthquake.

FIGURE 1
VISION 2000 PERFORMANCE OBJECTIVES



The single performance evaluation inherent in the current building codes is appealing to those responsible for developing, adopting and enforcing them as it aligns well with the basic role of public safety protection intended for these documents. However, because the Life Safety performance level is relatively poorly defined in terms of the margin against failure provided, this performance evaluation has little technical meaning. As our future codes move towards a more closely performance-based concept it would be preferable to abandon the so-called Life Safety basis for design and adopt an approach that is truer to LRFD methods. Specifically, as with all LRFD design methods, two performance states should be considered - a serviceability state, similar to the Operational level of the *NEHRP Guidelines*, and a failure, or collapse state. Structures should be proportioned such that they provide an appropriate margin against the collapse state under maximum expected, or considered, levels of load and such that they do not exceed the serviceability state under frequent levels of load.

The concept of gradation of performance objectives based on building occupancy and use, as suggested by *Vision 2000*, is an appropriate one. However it is not necessary to adopt four independent levels of performance, as suggested by *Vision 2000*, in order to attain the enhanced performance desired for such structures. The two basic LRFD levels, serviceability and collapse are sufficient for this purpose. For relatively more important structures, the design margin against the collapse state for maximum expected loads should be increased relative to ordinary structures. Similarly, for such important structures, the load level at which serviceability performance is required should also be increased, such that the probability that the serviceability level is exceeded, is reduced.

THE 1997 NEHRP PROVISIONS

The proposed *1997 NEHRP Provisions* (BSSC, 1997b.), under development by the Building Seismic Safety Council (BSSC), have taken an important first, though not complete, step in the direction proposed above. Earlier editions of the Provisions, for example the *1994 Provisions* (BSSC, 1995) followed the traditional approach of designating a single design level of earthquake ground motion, having a 10% chance of exceedance in 50 years, for which ordinary buildings were designed to provide Life Safety performance. Under these provisions, more important structures were designed for the same load levels, but were required to conform to stricter drift limits so as to reduce expected damage levels, in a non-quantitative manner, under the design ground motion.

In the development of the *1997 Provisions*, it was decided to abandon the concept of a design basis earthquake ground motion with uniform probability of exceedance throughout the nation. Instead, in the development of new hazard mapping for use by the Provisions, the United States Geologic Survey (USGS) was directed to depict ground motion response parameters for a maximum considered earthquake (MCE) ground motion. This MCE motion is typically defined as having a 2% probability of exceedance in 50 years, as it was deemed that consideration of less probable levels of ground motion would be inconsistent with the level of risk adopted by society with regard to other hazards. The exception to this definition of MCE motion is in areas close to known active faults capable of producing large magnitude events. In such locations, where the source of the hazard is well defined, it was felt that rather than resorting to a probabilistic definition of MCE ground motion it would be more appropriate to base MCE ground motion on a maximum, or characteristic, magnitude earthquake on the fault. Specifically, it is taken as 150% of the ground shaking obtained from median attenuation relationships for the characteristic earthquake. In essence, using the terminology discussed relative to LRFD approaches, the MCE ground motion is the maximum expected loading.

The intent of an LRFD approach is to design for a high confidence of a low probability of failure at maximum expected load. Determination of the probability of failure for a multi-degree of freedom, nonlinear structural system such as a building, in response to complex dynamic loading such as earthquake ground motion is an exceedingly difficult task and has never been performed in a comprehensive manner for the wide range of structural systems covered under the scope of the *NEHRP Provisions*. However, it was the judgment of members of the Seismic Design Procedures Group (SDPG), a joint task force of the BSSC and USGS engaged in development of the new hazard maps and provisions, that buildings designed and constructed in conformance with the procedures of the *1994 NEHRP Provisions* would be able to resist a loading at least 1½ times larger than the design ground motion, without collapse. Therefore, it was decided in the *1997 NEHRP Provisions* to specify a design ground motion (loading) that is 1/(1.5) or 2/3 of the MCE ground motion, such that for maximum expected loading (the MCE ground motion), a high confidence of a low probability of failure would exist.

Another important feature of the proposed *1997 Provisions* is the introduction of an occupancy importance factor, I, to regulate the amount of margin provided in a design depending on the use and importance of the building. The I factor is introduced into the base shear as a modifier on the response modification coefficient, R, such that an effective response modification coefficient

(R/I) is used to determine design force levels. The value of I varies from 1 for ordinary structures to 1.5 for essential structures. This has the effect of increasing the minimum presumed margin of 1.5 for ordinary structures to a value of $(1.5)^2$, or 2.25, for essential structures. In addition to increasing the inherent safety margin in essential structures, the I factor also has the effect of increasing the load level at which elastic behavior can occur and therefore results in a raising of the threshold level at which damage is expected to initiate, consistent with the design goals of providing reduced damage and improved safety in important structures.

FUTURE DEVELOPMENT

Although the proposed *1997 NEHRP Provisions* represent an important first step in the development of performance-based design approaches for new buildings, a significant amount of development yet needs to be performed. The proposed *Provisions* contain no specific requirement or procedure for evaluation of building serviceability. There is no basis, other than the collective judgment of the SDPG members, for the implied margins of safety. The design procedure used in the provisions still relies on linear structural analysis techniques, which inherently is based on an incorrect behavioral model for the structure at the failure state, and consequently incorporates large amounts of uncertainty. The Provisions have no procedure to account for important aspects of the loading function including duration and near-field velocity pulse effects.

The incorporation of a meaningful serviceability evaluation procedure should receive a high priority in future design procedure development. Recent California earthquakes have demonstrated a largely satisfactory reliability for our modern structures with regard to collapse avoidance, however, the failure of many modern structures to remain serviceable following moderate to severe ground shaking has been a source of concern to the business, financial and emergency management communities, and in fact, has largely lead to the current motivation for development of performance-based design procedures. Conceptually, it should be relatively simple to develop a serviceability evaluation procedure for future design provisions. Since by definition, serviceability implies limited damage, structures must behave in an elastic, or near-elastic manner at the service level and our existing linear analysis methods are probably adequate for the performance of a meaningful serviceability evaluation. However, considerable work must be performed to define an appropriate load level, or earthquake exceedance probability, at which serviceability should be obtained for structures of different occupancy and use. Further, much more research into the behavior of nonstructural building components including architectural mechanical and electrical components, is required to permit development of appropriate design parameters and acceptance criteria for these important building elements. Although the determination of acceptable levels of structural response and damage at a service level of performance is an engineering task, the most difficult task, determination of the service load level or exceedance probability is really beyond the sole province of the structural engineer and requires the participation of the financial, social planning and regulatory communities.

Development of reliable evaluation procedures for the failure, or collapse state, capable of accounting for the important velocity pulse and ground motion duration effects must also clearly occur. In addition, there is a need to develop a series of prototype, or model buildings, representative of our various structural systems, so that the reliability of various analytical and design procedures can be evaluated on a consistent basis. Finally, more comprehensive determination of the reliability inherent in our design procedures must be performed so that appropriate margins of safety can be maintained as our engineering procedures evolve.

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Displacement-Based Seismic Design: A Compromise Between Simple and Comprehensive

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SUMMARY

Great strides are being made to develop performance-based earthquake engineering methodologies for both new and existing construction. Displacement-based approaches have gained favor recently owing to their relative ease of application and their effectiveness as compared with traditional force-based approaches. Recent guidelines published by the Federal Emergency Management Agency and by the State of California Seismic Safety Commission describe performance-based earthquake engineering approaches for seismic rehabilitation of existing buildings. The basic approach of these guidelines is applied to an example building that was damaged in the 1994 Northridge earthquake. The application demonstrates the strengths and weaknesses of the displacement-based approach. The need for continued research to improve this approach and develop other approaches is emphasized.

INTRODUCTION

One of my earliest recollections of a "technical" discussion with my father came at the beginning of the 1960's. The discussion focused on the millennia-old dream of manned spaceflight, and the reckoned impossibility of landing a human on the moon and returning him safely to earth (Figure 1). At around the same time, unknown to me at the time, a young US president instituted the Apollo program designed for just that purpose, "before the decade is out." This national commitment resulted in an intensive, large-scale program to achieve its purpose. This interdisciplinary program was well timed in that it could build upon scientific findings such as those by Copernicus on the solar system, those by Newton on the laws of universal gravitation and motion, and those by Goddard on rocket propulsion. It was also well timed in the sense of urgency developed through international competition propelled by the cold war. The program to land a human on the moon was important in part because it succeeded in its ultimate objective, but more so because of the technologies that were spun off from the program, proving something else my father related to me: in sports as in many other activities, its how you play the game that is important; winning is only secondary.

Back on earth, earthquake engineering professionals today are grappling with the challenge of producing structures whose performance in future earthquakes is more predictable. The urgency in this program is the realization, made clear by recent earthquakes, that huge economic losses may result from urban earthquakes and that currently-available engineering approaches may not be meeting society's expectations. Success in performance-based earthquake engineering will rely on a commitment of financial and intellectual resources to the goal. As an interdisciplinary program, it must build upon and integrate scientific findings related to engineering seismology; geotechnical and structural engineering; and the social sciences including political, planning, and economics aspects. Today it is unclear whether we have reached the stage of technological development in the earthquake engineering field to make performance-based earthquake engineering a reality. But it will be good for our profession to try because of the technological advances it will engender.

As we move forward toward the goal of performance-based earthquake engineering, we must retain our focus on the role of research. As stressed by Professor Bertero on innumerable occasions, it is only through comprehensive and integrated analytical and experimental research, complemented by post-earthquake studies, that we will gain the understanding necessary to advance the state-of-the-art and the state-of-the-practice.

This paper presents only a small piece of that vast puzzle of performance-based earthquake engineering. Specifically, it focuses on the application of the displacement-based approach to seismic evaluation and rehabilitation of existing buildings. This approach forms the basis for recently-published performance-based earthquake engineering guidelines (FEMA 1996, SSC 1996). The approach will be applied to an existing seven-story reinforced concrete frame building that was damaged during the 1994 Northridge earthquake. The application includes both an evaluation of the structure in its as-built state and in a rehabilitated state

DESCRIPTION OF EXAMPLE BUILDING

The building is located in Van Nuys, California. The site is near the center of the San Fernando valley, approximately 4.5 miles from the epicenter of the 1994 Northridge earthquake. Originally designed in 1965, the approximately 65,000 square foot structure was built in 1966. Its primary use has been as a hotel, with restaurants, lobby, and services on the first floor. Details of the building are reported by Lynn (1996a).

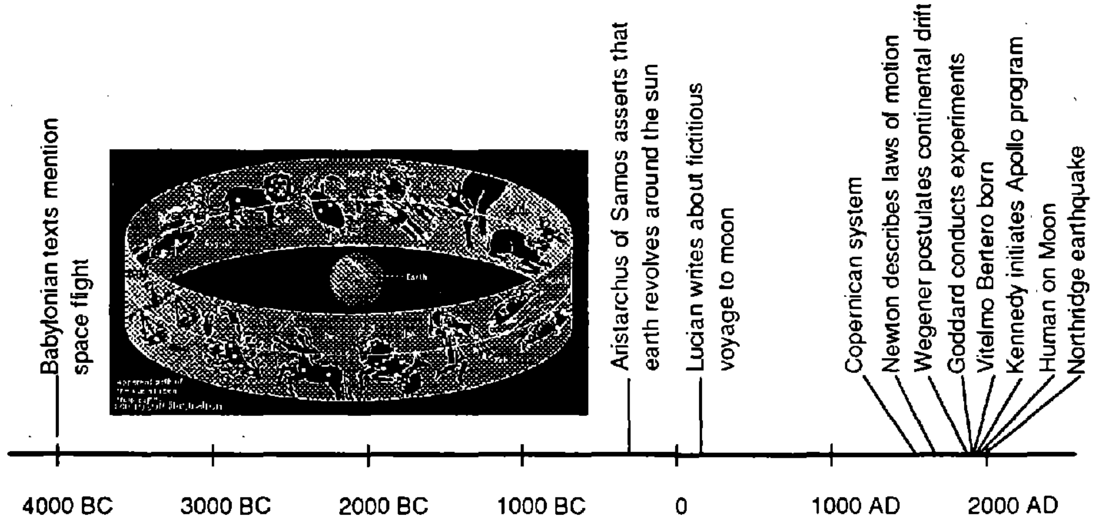


FIGURE 1 - A CHRONOLOGY OF IMPORTANT HISTORICAL EVENTS

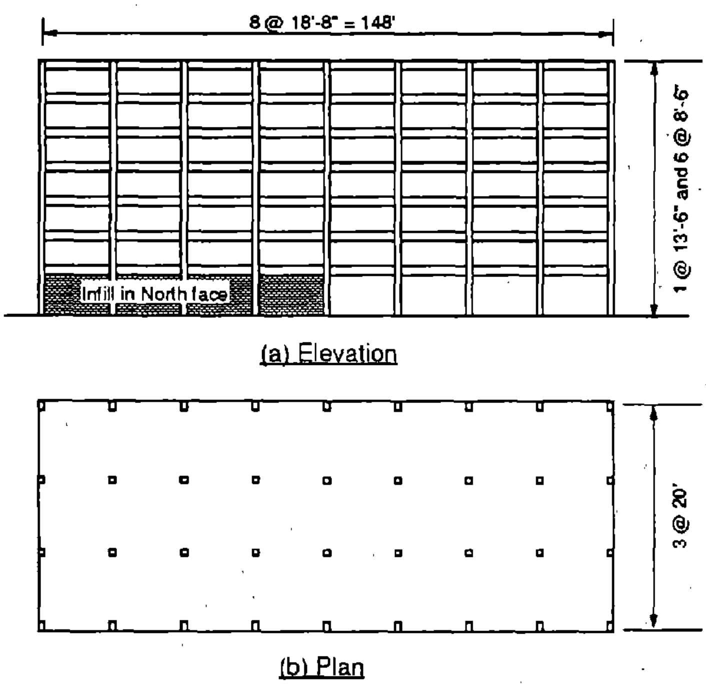


FIGURE 2 STRUCTURE ELEVATION AND PLAN

The building has a seven-story, reinforced concrete structure (Figure 2). The framing system comprises reinforced concrete perimeter beam-column framing and interior slab-column framing supported on reinforced concrete piles. The system is symmetric, with the exception of a brick infill in the four bays of the first floor of the north frame as shown. Expansion joints of nominal 1-inch and 0.5-inch gaps separate the infill from the sides and top of the frame, respectively. The building is supported on a friction pile foundation driven into primarily silty fine sands and fine sandy silts.

The exterior beam-column spandrel frames have details shown in Figure 3. As shown, the beam reinforcement details appear to nearly satisfy requirements for special moment frames. The 14 by 20 inch exterior columns do satisfy requirements for special moment frames because of inadequate tie configuration and spacing, and inappropriate length and location of lap splice. Beam-column joints lack joint ties required for special moment frames. The interior framing comprises reinforced concrete slab-column framing. The slab thickness typically is 8-1/2 inches and the column are 18 inches square. Materials are normal-weight aggregate concrete with design strength from 3000 to 5000 psi, grade 40 steel in beams and slabs, and grade 60 steel in columns.

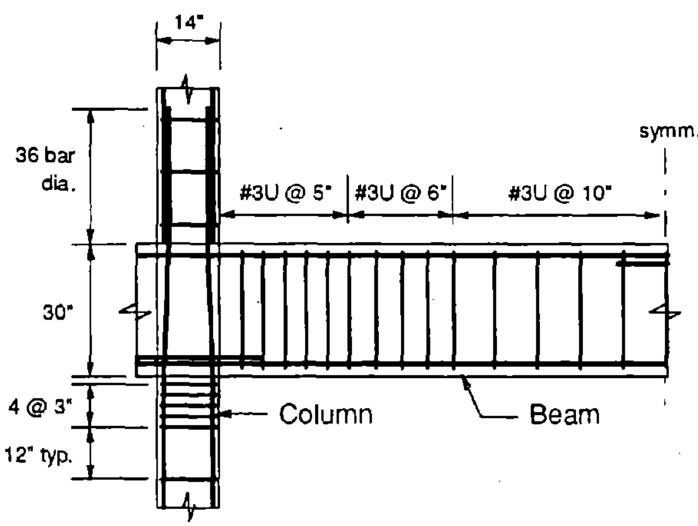


FIGURE 3 TYPICAL EXTERIOR FRAME DETAILS

DAMAGE SUSTAINED IN THE NORTHRIDGE EARTHQUAKE

During the 1994 Northridge earthquake, instruments at the ground floor recorded peak horizontal acceleration of 0.47g. The peak roof accelerations were 0.59g in the longitudinal direction and 0.58g in the transverse direction.

Damage sustained by the building is illustrated in Figure 4, which is a photograph taken after epoxy injection of cracks. Although not obvious in the photograph, several columns experienced severe damage at the top of the fourth story involving diagonal cracking, concrete spalling, reinforcement buckling, and bond splitting along the longitudinal reinforcement.

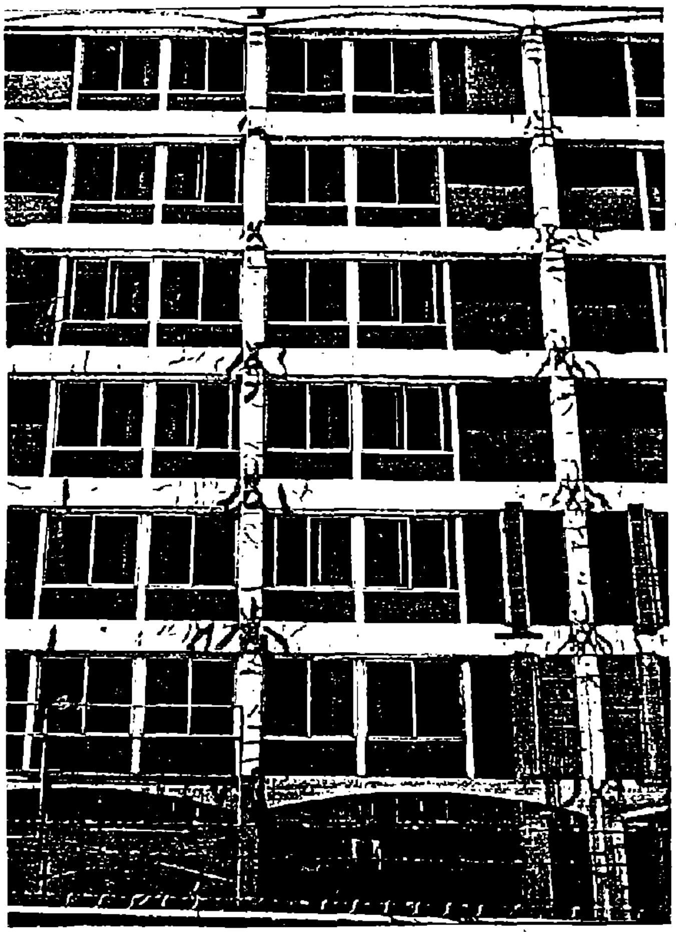
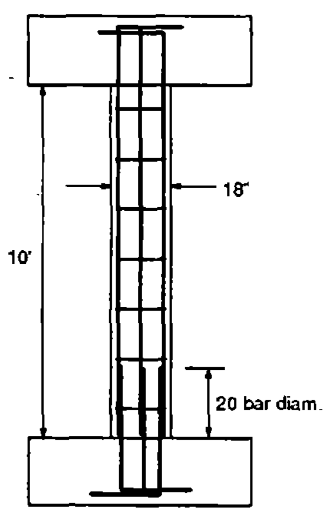


FIGURE 4 EPOXY REPAIR TRACES FOLLOWING NORTHRIDGE EARTHQUAKE

Lynn (1996b) reports test data for columns having widely-spaced perimeter hoops and lap splices. The columns were tested under moderate levels of axial load combined with lateral displacement cycles (the upper and lower foundation blocks were restrained against rotation). Figure 5 shows details and recorded load-deformation relation for one of the columns. The test data indicate that the moment and shear strengths were well represented by the design equations in ACI 318-95. Lightly-confined lap splices led to column hinging and shear failure; where splice failures occurred or where shear demands exceeded the ACI 318-95 nominal shear strength, the strength degraded relatively rapidly following yield. Loss of gravity load capacity generally followed shear failure. Therefore, for columns of with this detail it is necessary to avoid significant inelastic response.



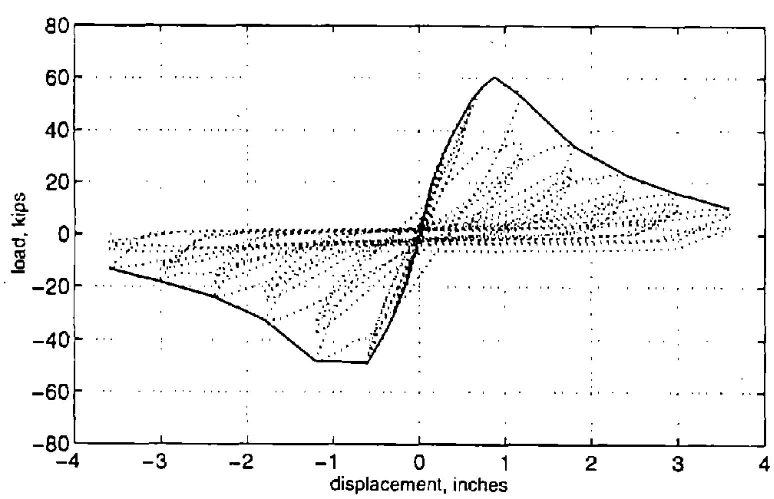


FIGURE 5 TEST SPECIMEN AND DATA

A DISPLACEMENT-BASED EVALUATION OF THE AS-BUILT STRUCTURE

The as-built structure (without the first-floor infill) was evaluated using the static inelastic evaluation approach of FEMA 273 (FEMA, 1996). The seismic hazard was represented using the response spectra of Figure 6. The Life-Safety Event is roughly equivalent to that obtained from the 1994 NEHRP provisions for new buildings, and corresponds to a 10% probability of exceedance in a 50-year period, while that for the Collapse-Prevention Event corresponds to a 2%/50yr event. (Note that new maps developed by the USGS and being considered for the 1997 NEHRP provisions may would result in higher spectral ordinates in coastal California.)

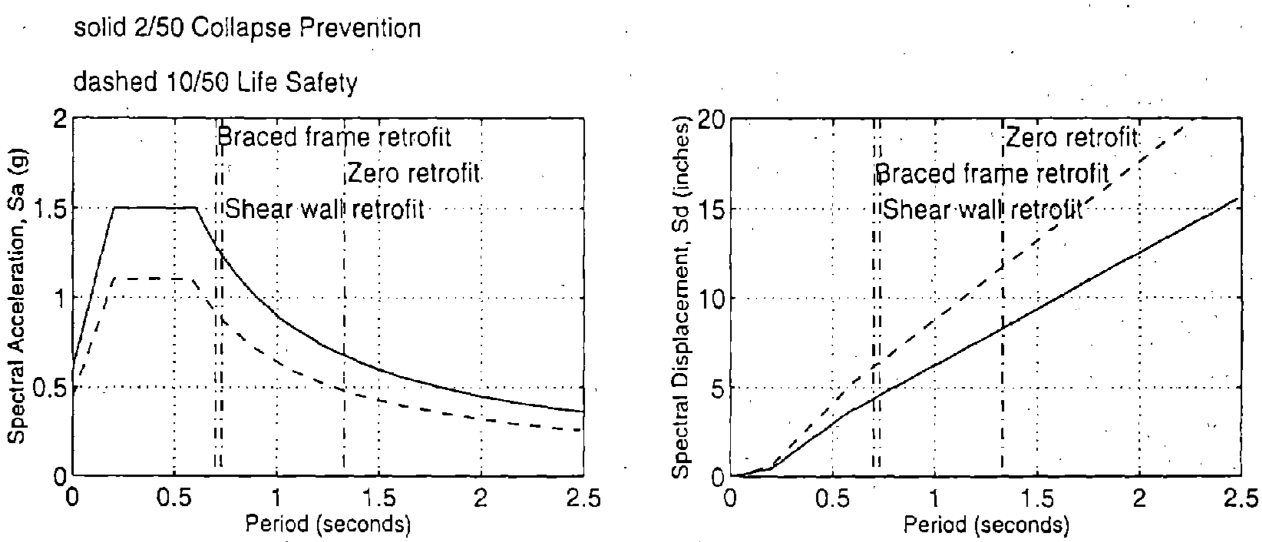


FIGURE 6 SPECTRAL DEMAND USED FOR THE EVALUATION AND DESIGN STUDY

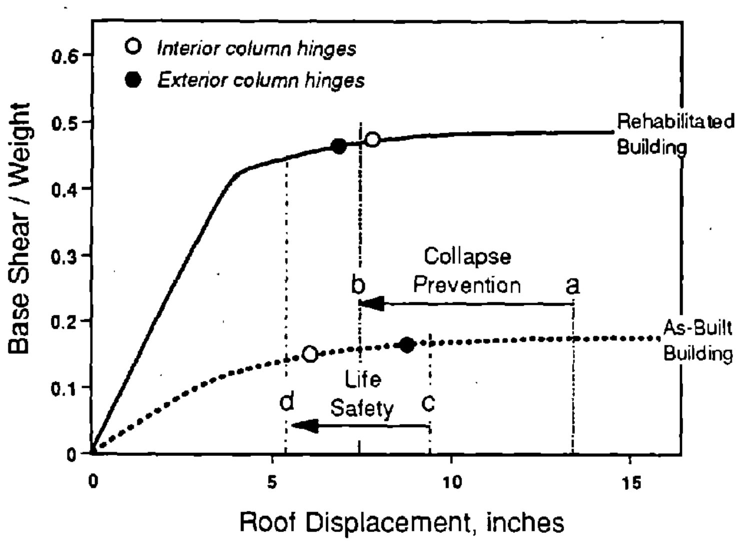


FIGURE 7 COMPUTED BUILDING LOAD-DEFORMATION RELATIONS

A two-dimensional numerical model of the building in the longitudinal direction was assembled using the program DRAIN-2DX (Prakash, 1993). The computed load-deformation relation is shown by the heavy broken curve in Figure 7. (The lateral loading profile had the same shape as that used for the equivalent static procedure of the NEHRP provisions. Results under an inverted triangular or uniform distribution were substantially the same.)

Given the long fundamental period of the building relative to the characteristic ground period

(Miranda, 1991), the displacement demand for design can be taken equal to the elastic spectral demand adjusted to the roof level. With this assumption, the Life-Safety evaluations (point "a") and (particularly) the Collapse-Prevention evaluations (point "c") result in calculated column demands well beyond the onset of flexural yielding. Given the details (short lap splices and widely-spaced transverse reinforcement), the available data suggest inadequate column and overall building performance for the design events. (Other aspects, such as possible degradation of the unconfined joints under extended ductility demands, are also suspect.)

A REHABILITATION APPROACH

A range of rehabilitation strategies is available for the subject structure. For each, a primary objective is to avoid relatively brittle inelastic response of the columns. One strategy would be to jacket the individual columns, thereby improving their flexural ductility, shear strength,and splice confinement. While this approach is technically feasible, it may be economically unfeasible because it would be required for all columns throughout the building height and would require considerable interior disruption. This approach would not significantly affect the building period or strength, so that the overall displacement demands would not be significantly altered from those indicated for the as-built structure in Figure 7.

An alternative strategy is to provide new structural elements to modify the building's dynamic characteristics and thereby reduce the demands on the vulnerable columns. These could be in the form of structural walls, steel bracing, seismic isolation, or added building damping. One possible solution is addition of new structural walls, as illustrated in Figure 8. The walls are positioned around the perimeter to avoid interior disruption, and are located as shown to ensure that each interior room has window space. The walls are proportioned primarily considering stiffness, the objective being to reduce the lateral displacements to a level that will not distress the columns. Referring to Figures 6 and 7, it is necessary to reduce the initial period to about three-quarters of a second to avoid column yielding at the target displacement level. The walls were proportioned considering this objective, and the structure was re-analyzed.

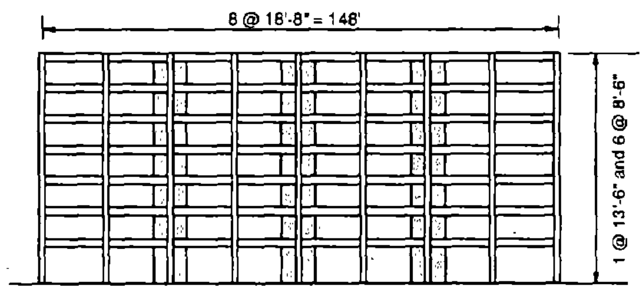


FIGURE 8 A REHABILITATION STRATEGY

The computed load-deformation relation for the conceptually-rehabilitated building is shown by the heavy solid curve in Figure 7. Also shown are the revised displacement demands (points "b" and "d") determined from the initial period and the design spectra of Figure 6. In the revised condition, the deformation demands on the columns, both for the exterior frames and the interior frames, are reduced to acceptable levels. Other aspects of the proposed scheme, including demands on the shortened spandrel beams and the foundations, must be checked as part of rehabilitation design.

DISCUSSION

The focus on displacements, as opposed to the traditional strength-based approaches, offers distinct advantages. Because of the sensitivity of the columns to inelastic deformations, it is a relatively simple process to determine the necessary characteristics of the rehabilitated building, in terms of stiffness (and possibly strength) to control the deformations. Both the primary lateral force resisting elements (new and old) and the secondary elements can be checked in a single step, as displacement demands are postulated for both. Force-based or ductility-based approaches do not provide direct information on performance of those elements that do not contribute significantly to lateral force resistance.

Two fundamental assumptions that have been made should be exposed. First, it has been assumed that the displacement demands can be estimated in a useful way. Additional research is needed to better quantify ground motion characteristics as well as structural responses to these ground motions; both aspects are current subjects of in-depth research and hot debate. Second, it has been assumed that component deformation capacities can be estimated. More studies are needed to quantify the inelastic response of older existing construction, including studies of effects of multiple cycles of loading.

CONCLUSIONS

Performance-based earthquake engineering offers the opportunity to design structures of more predictable performance over a broader range of performance objectives. It also challenges the engineering practice and research professions to develop, verify, and implement new procedures. Displacement-based procedures are one of several viable approaches that have been put forth. An example is used to demonstrate its basic approach and to suggest its validity. The example also serves to reinforce the need for continued research to further improve the approach.

ACKNOWLEDGMENT

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Seismic Design Based on Inelastic Behavior

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SUMMARY

Performance based seismic design implies that we have to predict - quantitatively - the behavior of structures in postulated design earthquakes and compare predicted performance to acceptable performance. An efficient way to achieve acceptable performance is to incorporate all important performance objectives already in the conceptual design phase. Focusing on low performance levels, e.g., collapse prevention, the response of structures will be controlled by inelastic behavior characteristics. A relatively simple approach to incorporating these characteristics in the conceptual design and performance evaluation phases of the design process is illustrated in this paper.

INTRODUCTION

In the simplest case, seismic design can be viewed as a two phase process. The first, and usually most important one, is the conception of an effective structural system that needs to be configured with due regard to all important seismic performance objectives, ranging from serviceability considerations to life safety and collapse prevention. This phase, referred to here as the conceptual design phase, comprises the art of seismic engineering since no rigid rules can, or should, be imposed on the engineer's creativity to devise a system that not only fulfills important seismic performance objectives but also pays tribute to functional and economic constraints imposed by the owner, the architect, and other professionals involved in the design and construction of a building. By default, this process of creation needs to be based on judgment, experience, and a comprehensive understanding of seismic behavior, rather than on rigorous mathematical formulations. Rules of thumb for strength and stiffness targets, based on fundamental knowledge of ground motion and elastic and inelastic dynamic response characteristics, should suffice to configure and rough-size an effective structural system.

Elaborate mathematical/physical models can be built only once a structural system has been created. Such models are needed to evaluate seismic performance of an already created system and to modify component behavior characteristics (strength, stiffness, deformation capacity) to better suit the specified performance criteria. This second phase in the design process should involve a demand/capacity evaluation at all important performance levels, which requires identification of important capacity parameters and prescription of acceptable values of these parameters, as well as prediction of the demands imposed by ground motions. Suitable capacity parameters and their acceptable values, as well as suitable methods for demand prediction will depend on the performance level to be evaluated. This phase in the design process is referred to here as the performance evaluation and design modification phase.

This paper addresses fundamental issues in both design phases, but focuses on inelastic design considerations. Because of this focus the discussion is concerned only with low performance levels such as life safety and collapse prevention. Design considerations at higher performance levels are not addressed here, but it needs to be pointed out that their economic impact may equal or exceed that of life safety and collapse prevention design.

The concepts discussed here are based on the premise that an explicit design for different performance levels, including explicit considerations of inelastic behavior, will result in "better" designs than the present prevailing design approach in which all design considerations are lumped into one simplified elastic design procedure. The word better implies that the expected behavior will be more in tune with the desires expressed by owners, users, and society, and that designs can be rationalized sufficiently by first principles to satisfy the owner's desire for sound judgment on the costs and benefits of earthquake protection, and society's needs for informed decision making in the face of uncertain (and often unknown) seismic demands and equally uncertain (and often unknown) seismic capacities of existing and even new man-made construction.

The last sentence is easy to put on paper but difficult to digest and implement. In fact, its implementation is a dream far from reality - for many reasons of which the following two are prevalent: (1) we don't know enough about the seismic input and the capacities of structures and their elements, and (2) even if we would know, we don't have adequate models to capture all important phenomena (e.g., damage accumulation under random loading) and their uncertainties to draw well informed conclusions. With this confession before us, we can take one of two actions. One is to do nothing and pretend that the present oversimplified design approach will do just fine (the ostrich head in the sand approach), and the other is to be daring and venture into the scary and more complex world of the unknown and try to come closer to the truth (the crusader or Don Quixote approach, depending on the perspective). The second one appears to be preferable.

PERFORMANCE ISSUES AT LIFE SAFETY / COLLAPSE PREVENTION LEVELS

The terms life safety and collapse prevention have become standard terms in the performance based design world. There are specific definitions to go along with these terms, as for instance those contained in FEMA, 1996, and SEAOC, 1995. In essence, the definitions imply that no life threatening falling hazards should be present in a life safety event, and that all critical components of the gravity load resisting system must maintain sufficient gravity load carrying capacity in a collapse prevention event. Deterioration in strength and stiffness are acceptable behavior modes unless they lead to an unacceptable reduction in the gravity load carrying capacity or to excessive lateral deflections that invite incremental collapse (dynamic P-delta instability).

Deterioration may occur at a rapid rate (e.g., fracture at a welded connection) or a slow rate (e.g., local buckling in a steel beam flange). Clearly, the consequences, at least at the element level, are very different between the two modes. The problem with predicting deterioration is that it depends strongly on the deformation history to which an element has been subjected. This history in turn depends on the ground motion and structural response characteristics. In case of a near-field pulse type earthquake the response of an element could be of the type shown in Fig. 1(a), and in case of a more distant but long duration earthquake the response could be of the type shown in Fig. 1(b). The deterioration behavior, and therefore the performance of the element, will be very different in the two cases.

Several empirical models for deterioration prediction have been proposed in the literature. In concept, most of these models assume that element deformation capacity and deterioration can be expressed as a function of the energy dissipated in an excursion (or cycle). This is a good reason to advocate energy based design. Concepts and approaches for energy based design can be found in the literature (Bertero et al., 1996, , Fajfar et al, 1992), and are discussed at this symposium.

Until energy based design concepts, or other concepts that account explicitly for deterioration, take a stronger foothold, we will have to live with simplified approaches in assessing acceptable performance in design for low performance levels. If we accept the notion that deformations and not forces control the behavior of ductile elements, then an allowable deformation (displacement, rotation angle, angle of distortion, etc.) needs to be assigned to each element. Looking at Fig. 1, it is evident that the choice of a single value, denoted with δ_{all}, will have to be based on judgment. To make matters worse, for most elements we don't even know with good confidence how the load-deformation response looks like.

The previous paragraph invites the argument that prediction of inelastic behavior is irrelevant, if acceptable deformation values are so poorly defined. This argument misses the point because there is much more to design based on inelastic behavior than analytical prediction of deformation demands at the element level. An accurate prediction is desirable but not critical, particularly for elements that deteriorate in a gradual manner. What is more important is the realization that life safety hazards are caused primarily by brittle failure modes in components and connections that are important parts of the gravity and lateral load paths. Thus, the emphasis in design for life safety and collapse prevention needs to be on

- providing a sound load path and making sure that the load path remains sound at the maximum deformations expected in a design earthquake,
- providing connections between elements that remain capable of transferring loads between the elements that form part of the load path,
- providing sufficient strength and stiffness (together with ductility) so that deformation demands do not become excessive to the extent that incremental collapse (global dynamic P-delta instability) can occur,
- avoiding overloads on elements that may fail in a brittle mode and that are important parts of the load path, and
- making sure that localized failures (should they occur) do not pose a collapse or life safety hazard, i.e., that the loads tributary to the failed element(s) can be transferred safely to other elements and that the failed element itself does not pose a falling hazard.

In many cases these objectives can only be achieved if inelastic behavior is accounted for explicitly. Good design is a matter of judging (predicting) the consequences of element deterioration on system behavior and taking proper action if these consequences endanger the vertical load carrying capacity of the system.

CONCEPTUAL DESIGN

Conceptual design implies configuring and rough-sizing a structurally and economically effective system which, when later subjected to the performance evaluation and design modification process, will come close to fulfilling all important performance criteria. Design modification will then be limited to fine tuning of member sizes and detailing requirements.

Perhaps the most useful tools for conceptual seismic design at low performance levels are design displacement spectra. There are two reasons for this. First, in most cases, displacements will control the design. Element deformations (e.g., plastic hinge rotations in beams) are a function of story drift, which in turn is related to the global displacement of the structure. Moreover, even at low performance levels story and global drift control may by critical because of incremental collapse (dynamic P-delta) protection. Second, in general the expected roof displacement of a structure can be predicted with good accuracy from the displacement spectrum, which in turn implies that global stiffness requirements can be derived from spectral displacement demands.

For regular structural systems (no large vertical or torsional irregularities) much data exists to relate roof displacement of a structure to SDOF spectral displacement. Displacement response is usually controlled by first mode vibrations, even for long period structures. In a statistical study using 15 typical soil type S_1 ground motion records it was found that for both elastic wall structures (controlled by a global flexural mode of deformation) and elastic frame structures (shear mode of deformation) the roof displacement, $\delta_{t,el}$, can be predicted with good accuracy as the first mode SDOF spectral displacement, $\delta_{SDOF,el}$, times the first mode participation factor (Seneviratna, 1995). The study by Seneviratna has also shown that the ratio of roof displacements of inelastic and elastic MDOF structures ($\delta_{t,in}/\delta_{t,el}$) is rather stable and is as shown in Fig. 2. This ratio depends somewhat on the global ductility ratio (denoted as μ(SDOF)) at longer periods and strongly on μ(SDOF) at short periods, and it is not vastly different from the ratio δ_{in}/δ_{el} of the first mode SDOF system (see Fig. 3). Furthermore, statistical data is available from the same study that relates maximum interstory drift ($\delta_{s,max}/h_i$) to the global drift (δ_t/h_t) for different types of frame structures. For instance, for frame structures in which the relative story shear strength is tuned to a code type shear force pattern and in which plastic hinging is limited to beams only, the mean values of the ratio of maximum interstory drift to global drift are shown in Fig. 4.

The utility of the just presented information for conceptual design can be illustrated with the following example. A 10 story steel moment resisting frame is to be designed in California. The height of each story is estimated as 13 feet. It is assumed that the collapse prevention design spectrum in the constant velocity region is represented by the equation $S_a = 0.6g/T$. As a basic design criterion it is stipulated that the interstory drift should not exceed a value of 0.02. How can this criterion be used to derive global stiffness (and strength) requirements for the frame structure?

The spectral displacement is given as $S_d = (T^2/4\pi^2)S_a = (T^2/4\pi^2)(0.6g/T) = 5.9T$ (in.).

Using a first mode participation factor of 1.4, the roof displacement of the elastic structure is estimated as $\delta_{t,el} = 5.9T\times 1.4 = 8.2T$.

Figure 2 indicates that the ratio $\delta_{t,in}/\delta_{t,el}$ is rather insensitive to T and the extent of inelasticity (μ(SDOF)) in the range of T between 1.5 and 2 sec. For a global ductility (μ(SDOF)) around 4 a good estimate of $\delta_{t,in}/\delta_{t,el}$ is 0.75, and therefore a good estimate of roof displacement of the inelastic structure is $\delta_{t,in} = 0.75\times 8.2T = 6.2T$.

From Fig. 4 it can be seen that the ratio of maximum interstory drift to global drift is sensitive to the first mode period, and an estimate of T (and μ(SDOF)) need to be made. Estimating T = 2.0 sec. and μ(SDOF) = 4, the ratio $(\delta_{s,max}/h_i)/(\delta_t/h_t)$ can be estimated as 2.8, which implies that the global drift index should be limited to 0.02/2.8 = 0.0071. Using this limit, i.e., $\delta_{t,in} = 6.2T \geq 0.0071(130\times 12) = 11.1$ in., shows that the first mode period of the structure should be limited to 11.1/6.2 = 1.8 seconds. This provides a clear stiffness requirement for design, which in fact is more stringent than required by present codes.

One can carry the arguments one step further and derive also approximate strength design requirements. The stiffness requirement was derived assuming a global ductility (μ(SDOF)) of 4. As Fig. 4 shows, the drifts are somewhat but not very sensitive to this global ductility target. Thus, strength is somewhat but not very relevant, provided, however, that strength is distributed rationally over the height of the structure, i.e., in proportion to a rational code, or better, SRSS shear force pattern.

An estimate of the base shear strength requirement (at the structure strength level, and not element strength level) can be obtained from an inelastic strength demand spectrum or an elastic spectrum and appropriate strength reduction factor, R. An excellent summary of R-factor approaches is presented in Miranda and Bertero, 1994. Figure 5 presents mean R-factors for the records employed in the study on which Figs. 2 to 4 are based. For $\mu = 4$ and T = 1.8 sec., the estimate of the mean R-factor is 5.2. Using the elastic design spectral acceleration of 0.6g/T, and a first mode effective mass of 80%, the base shear demand can then be estimated as $V = (0.6/1.8)\times 0.8W/5.2 = 0.051W$. This structure strength demand is rather low and it is likely that strength design will be controlled by a higher performance level.

The data on which Figs. 2 to 5 are based have been derived for nondeteriorating bilinear systems with zero strain hardening. The art of engineering enters when estimates have to be made of the effects of deterioration and of negative post-yield stiffness (due to P-delta effects) on the global and interstory drift demands. These effects may be estimated in the conceptual design phase, but are likely better suited for incorporation in the performance evaluation and design modification phase as is done in this discussion.

After global stiffness and strength demands have been estimated, member sizing can proceed in a customary fashion, design evaluation can be performed as discussed in the next section, and detailing can proceed with a focus on the areas in which the performance evaluation shows the highest demands. Compared to presently employed procedures, large differences are noted in the estimation of global demands in which the empirical code base shear equation is replaced by an approximate but transparent estimate of global stiffness and strength demands, and in the design evaluation phase in which efforts are being made to obtain realistic estimates of the force and deformation demands expected in each structural element.

PERFORMANCE EVALUATION AND DESIGN MODIFICATION

Once a structural system has been created, its performance must be evaluated as rigorously as feasible and design modifications may be needed in order to comply with specified performance requirements. In an ideal world there would be no debate about the proper method of demand prediction and performance evaluation at low performance levels. Clearly, inelastic time history analysis that predicts with sufficient reliability the bounds on forces and cumulative deformation (damage) demands in every element of the structural system is the final solution. Implementation of this solution requires the availability of a set of ground motion records (each with three components) that account for the uncertainties and differences in severity, frequency characteristics, and duration due to rupture characteristics and distances of the various faults that may cause motions at the site. It requires further the capability to model adequately the cyclic load-deformation characteristics of all important elements of the three-dimensional soil-foundation-structure system, and the availability of efficient tools to implement the solution process within the time and financial constraints imposed on an engineering office. Moreover, it requires adequate knowledge of element deformation capacities with due regard to deterioration characteristics that define the limit state of acceptable performance.

We need to work towards this final solution, but we also need to recognize the limitations of today's states of knowledge and practice. It is fair to say that at this time none of the aforementioned capabilities has been adequately developed and that efficient tools for implementation do not exist. Recognizing these limitation, the task is to perform an evaluation process that is relatively simple but captures the essential features that significantly affect the performance goal. In this context, accuracy of demand prediction is desirable, but it may not be essential since neither seismic input nor capacities are known with accuracy. The inelastic static pushover analysis, which is briefly summarized here, serves this purpose provided its limitations and pitfalls are fully recognized.

In the pushover analysis the structure is represented in a two- or three-dimensional analytical model that accounts for all important linear and nonlinear response characteristics, lateral loads are applied in predetermined patterns that represent approximately the relative inertia forces generated at locations of substantial masses, and the structure is pushed under these load patterns to specific target displacements. A target displacement is a characteristic displacement in the structure that serves as an estimate of the global displacement experienced by the structure in a design earthquake associated with a specified performance level. The internal forces and deformations computed at the target displacement levels are estimates of the strength and deformation demands, which need to be compared to available capacities.

The target displacement at which evaluation is to be performed can be determined in many different ways. One way is use the design spectral acceleration at the first mode period, S_a, as the basis, obtain the elastic spectral displacement, and modify this value for various phenomena that may significantly affect the roof displacement of the structure. FEMA 273 uses such an approach.

Thus, the target displacement can be expressed by the equation

$$\delta_{t,t} = (\prod C_i)\frac{T_1^2}{4\pi^2}S_a \tag{1}$$

where $\prod C_i$ is the product of modification factors accounting for important phenomena, such as the following:

Modification for MDOF effects. This factor transforms the equivalent SDOF displacement to the building roof displacement. It may be taken equal to the first mode participation factor.

Modification for yield strength. This factor accounts for the difference between displacements of inelastic and elastic systems. Even though not quite correct, this factor is often taken from SDOF spectral information, i.e., it is estimated from the ratio δ_{in}/δ_{el} of SDOF spectra (see Fig. 3).

Modification for stiffness degradation. Simulation studies on SDOF systems indicate that stiffness degradation that leads to pinching of hysteresis loops has little effect on the displacement demand, except for very short period systems. Thus, the need for such a modification is still in question.

Modification for strength deterioration. Strength deterioration may have a significant effect on the inelastic displacement demand. Unfortunately there is no simple answer to the magnitude of its effect, which depends strongly on the rate of deterioration and the strong motion duration of the ground motion. More research on this subject is urgently needed.

Modification for P-delta effect. Structure P-delta effect (caused by gravity loads acting on the deformed configuration of the structure) will always lead to an increase in lateral displacements. If P-delta effect causes a negative post-yield stiffness in any one story, this story may drift (increase in displacements in one direction), which may affect significantly the interstory drift and the target displacement. For SDOF systems the P-delta effect has been studied in some detail (Rahnama and Krawinkler, 1993). An example of its influence on the displacement demand of inelastic SDOF systems is shown in Fig. 6. More work needs to be done with MDOF systems in order to assess to what extend the SDOF information can be generalized to the MDOF domain.

Other modifications. There are several other effects that could be considered as modifications to the target displacement, including effects of different viscous damping, effects of foundation flexibility, torsional effects, etc.

In the writer's opinion too many recent arguments have focused on "exact" quantitative assessment of all these modifications rather than on the more fundamental issues associated with the pushover analysis. These issues have to do with the selection of load patterns that will bound the range of expected behavior of complex structures, and with a clear identification of the limitations of this simplified evaluation approach. Some of these issues are discussed in Krawinkler, 1997.

CONCLUDING REMARKS

This paper touches on only a few points of the emerging field of performance based design. The term "performance based" necessitates quantitative prediction of performance, which implies that we no longer can hide behind a book of rules that masks the physical concepts that control the seismic response of a structure. This paper attempts to point out why inelastic response considerations should be part of the design process and provides some suggestions how these considerations could be incorporated in the conceptual design and performance evaluation phases. Up front, it is pointed out that the present state of knowledge in quantifying element deformation behavior is rudimentary and that presently available methods for considering inelastic response in the design/evaluation process need much improvement. In other words, we have a long road ahead before performance based design can become a widely accepted engineering approach.

But we have made some headway in our efforts, mostly because a few individuals have made it their crusade to push for progress and lead the way with pioneering research efforts. Looking at the last 25 years, there are few if any who can match the contributions Professor Bertero has made to the understanding and advancement of seismic behavior and design. His work has paved the way for many of his students and for others to advance the state of knowledge and become contributors in their own right. I am proud to be one of these individuals who, albeit long time ago, has taken his fundamental in-depth lessons in earthquake engineering from the master.

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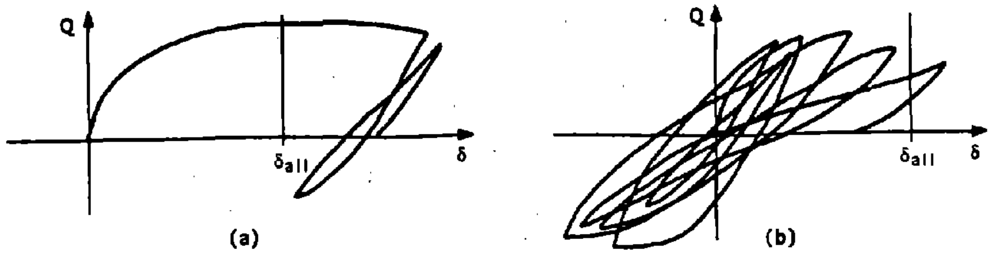


FIG.1 HISTORY DEPENDENCE OF DETERIORATION

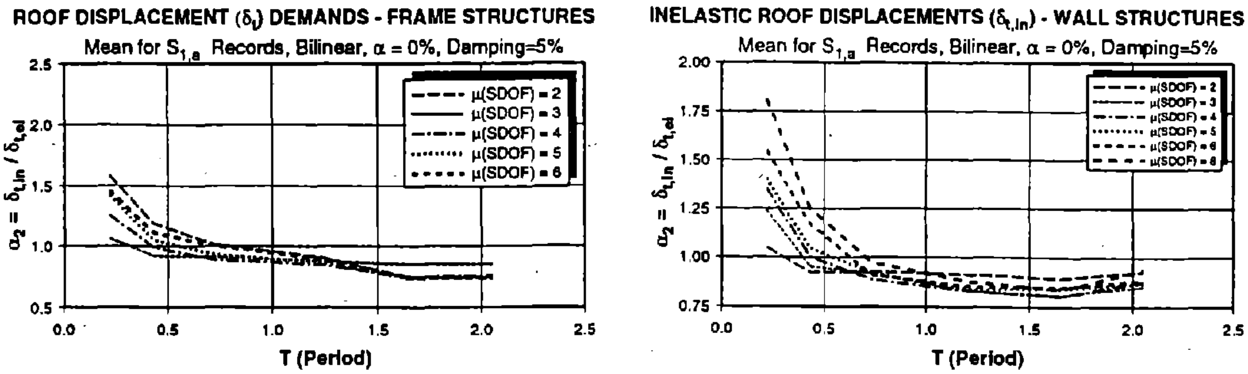


Fig. 2 RATIO OF INELASTIC TO ELASTIC ROOF DISPLACEMENT OF MDOF STRUCTURES

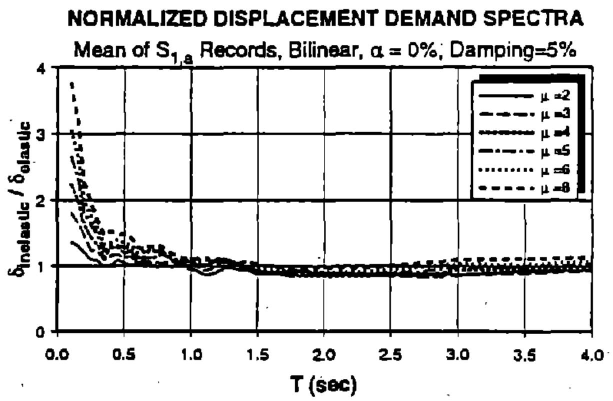


FIG. 3 RATIO OF INELASTIC TO ELASTIC DISPLACEMENTS OF SDOF SYSTEMS

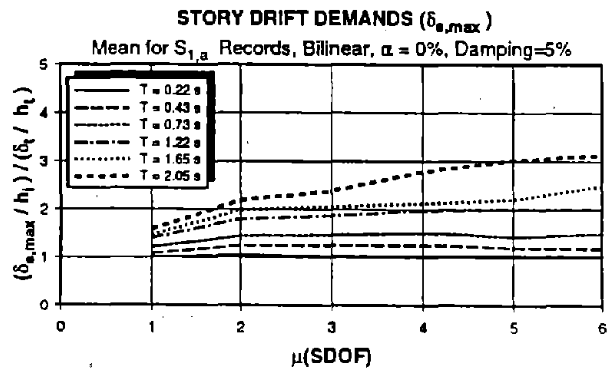


FIG. 4 RATIO OF MAXIMUM INTERSTORY DRIFT TO GLOBAL DRIFT FOR FRAME STR.

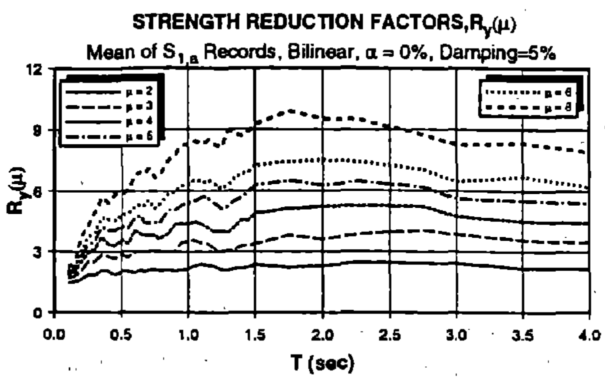
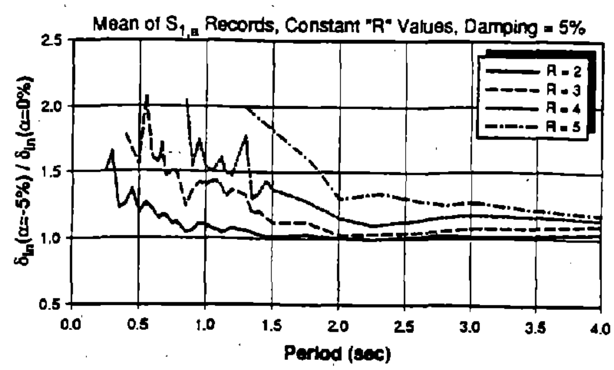


FIG. 5 STRENGTH REDUCTION FACTOR FOR BILINEAR SDOF SYSTEMS (S_1 SOIL)



Bilinear Systems with 5% Negative Stiffness

FIG. 6 EFFECT OF NEGATIVE POST-YIELD STIFFNESS ON DISPLACEMENT DEMAND

Strength Reduction Factors in Performance-Based Design

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SUMMARY

Strength reduction factors that are used to reduce design forces in earthquake resistant design are discussed. Based on recent research, the paper presents the different components of the so called *R* factors and discusses how these can be incorporated into a performance-based earthquake resistant design. The first component discussed is the reduction in lateral strength demand produced by nonlinear behavior in the structure which takes into account the hysteretic energy dissipation capacity of the structure. The paper presents first a summary and comparison of recent statistical studies on strength reduction factors computed for single-degree-of-freedom systems undergoing different levels of inelastic deformation when subjected to a large number of recorded earthquake ground motions. Despite having used significantly different ground motions data bases, results from various studies are remarkably similar. The main parameters that affect the amplitude of strength reductions are discussed. The evaluation of the results indicates that strength reductions due to nonlinear behavior are primarily influenced by the maximum tolerable displacement ductility demand, the period of the system and the soil conditions at the site. Based on these parameters simplified expressions that can be used in codes are presented. The paper then describes how strength reduction factors derived from single-degree-of-freedom systems need to be modified in order to be used in the design of multi-degree-of-freedom systems. Reductions in design forces due to overstrength are discussed. These reductions are due to the fact that the lateral strength of a structure is typically higher and in some case much higher than the nominal strength capacity of the structure. These reductions can be divided to take into account the additional strength from the nominal strength to the formation of the first plastic hinge and the additional strength from this point to the formation a mechanism. Finally, the paper discusses how these reductions factors can be implemented in performance-based design.

INTRODUCTION

Design lateral strengths prescribed in earthquake-resistant design provisions are typically lower and in some cases much lower than the lateral strength required to maintain a structure in the elastic range in the event of severe earthquake ground motions. Strength reductions from the elastic strength demand are commonly accounted for through the use of reduction factors. In U.S. practice the reduction factors are called *response modification factor, R,* in the National Earthquake Hazard Reduction Program (NEHRP, pp. 35-39) or *system performance factor,* R_w, in the Uniform Building Code (UBC-1988) and the Structural Engineers Association of California (SEAOC-1988). While reduction factors prescribed in seismic codes intent to account for damping, energy dissipation capacity as well as for overstrength, the level of reduction specified in seismic codes is primarily based on the observation of the performance of different structural systems in previous strong earthquakes. Strength reduction factors are one of the most controversial aspects of current buildings codes. Several researchers have expressed their concern about the lack of rationality in current R factors and their improvement has been identified as a way to improve the reliability of present earthquake-resistant design provisions (Bertero, 1986, Uang 1991, ATC, 1995b).

COMPONENTS OF STRENGTH REDUCTION FACTORS

REDUCTIONS DUE TO NONLINEAR BEHAVIOR

One of the first and better studied components of the R factors is the reduction in strength demand due to nonlinear hysteric behavior in a structure. The component of the strength reduction factor due to nonlinear hysteretic behavior, R_μ, is defined as the ratio of the elastic strength demand to the inelastic strength demand,

$$R_\mu = \frac{F_y(\mu = 1)}{F_y(\mu = \mu_i)} \qquad (1)$$

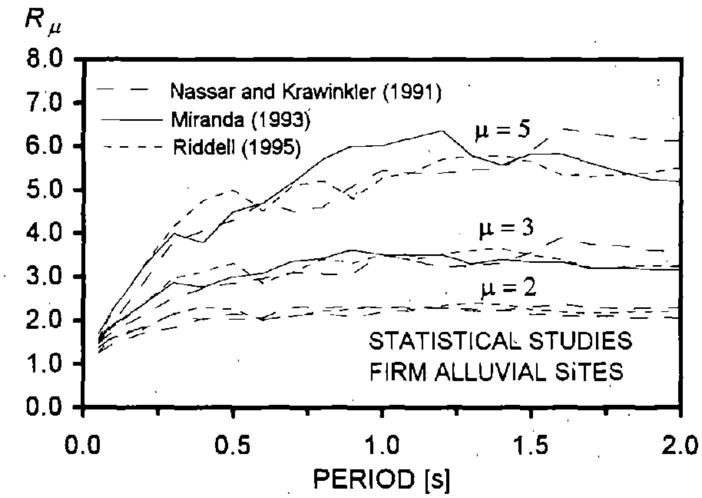
where F_y $(\mu=1)$ is the lateral yielding strength required to maintain the system elastic; F_y $(\mu=\mu_i)$ is the lateral yielding strength required to maintain the displacement ductility demand μ less or equal to a predetermined maximum tolerable displacement ductility ratio μ_i.

In general, for structures allowed to respond nonlinearly during earthquakes ground motions, inelastic deformations increase as the lateral yielding strength of the structure decreases (or as the design reduction factor increases). For a given ground motion and a maximum tolerable displacement ductility demand μ_i, the problem is to compute the minimum lateral strength capacity F_y $(\mu=\mu_i)$ that has to be supplied to the structure in order to avoid ductility demands larger than μ_i. Alternatively, for a given elastic design spectrum, the problem is to compute the maximum strength reduction factor that can be used in order to avoid ductility demands larger than μ_i.

For design purposes, R_μ corresponds to the *maximum* reduction in strength that can be used in order to limit the displacement ductility demand to a maximum tolerable ductility demand μ_i in a single-degree-of-freedom (SDOF) system that will have a lateral strength equal to the design strength.

For a given ground motion, computation of F_y $(\mu=\mu_i)$ involves iteration, for each period and for each target (i.e., maximum tolerable) ductility ratio, on the lateral strength F_y until the computed ductility demand μ is, within a certain tolerance, the same as the target ductility ratio μ_i. For a given ground acceleration time history, a R_μ spectrum can be constructed by plotting the strength reduction factors (computed with Eq. 1) of a family of SDOF systems (with different periods of vibrations) undergoing different levels of inelastic deformation μ_i when subjected to the same ground motion.

FIGURE 1.
COMPARISON OF MEAN STRENGTH REDUCTION FACTORS FOR FIRM SITES



Miranda and Bertero (1994) recently summarized the results of 13 different studies on strength reduction factors due to nonlinear behavior carried out in the last 30 years and put them in a common format in order to facilitate their direct comparison. A comparison of mean strength-reduction factors for systems subjected to ground motions recorded on firm alluvium sites from three different studies (Nassar and Krawinkler, 1991; Miranda, 1993; Riddell, 1995) is shown in **Figure 1**. The curve obtained by Nassar and Krawinkler was computed with 15 ground motions recorded in firm sites in California, Miranda's curve is computed from 62 ground motions recorded on firm alluvium sites in several countries during different earthquakes, while Riddell's curve is computed from 34 ground motions recorded on firm sites in Chile primarily during the march 3, 1985 earthquake. Although these studies are based on different sets of ground motions, the similarity of the results is remarkable and suggests that the general trends in reduction factors due to nonlinear behavior do not change significantly from one seismic region to another.

Based on the results of a comprehensive statistical study on strength-reduction factors of SDOF systems undergoing different levels of inelastic deformation when subjected to 124 ground motions, Miranda (1993) proposed simplified expressions to obtain analytical estimates of the strength-reduction factors for rock, alluvium and soft soil sites. Similarly, Nassar and Krawinkler (1991) and Riddell have recommended simplified expressions. However, none of these expressions have been incorporated into code provisions. Miranda's study showed that although some differences exist between strength reduction factors for rock and firm alluvium sites, for practical applications these differences are relatively small and can be neglected. If one makes such simplification and in the absence of more specific information on site conditions one could use the following simplified expression in the design of structures built on rock or firm sites:

$$R_\mu = \mu + (1 - \mu)\exp\left(\frac{-16\,T}{\mu}\right) \tag{2}$$

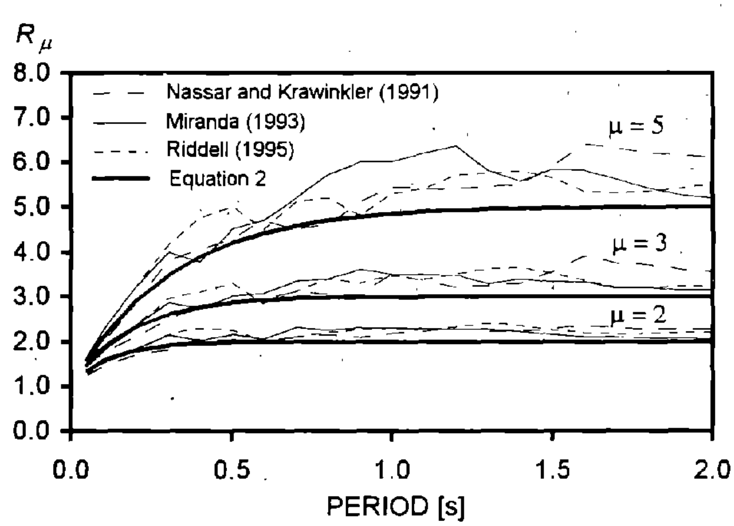
where μ is the displacement ductility ratio and T is the period of vibration. Equation 2 is simpler than the equations previously proposed and as shown in **Figure 2** has very good agreement with the statistical studies being only slightly more conservative.

For very soft soil sites, the shape of the R_μ spectrum is significantly different to that of rock and firm sites and strongly dependent on the predominant period of the ground motion T_S. Studies by Miranda have shown that the elastic and inelastic response of structures on very soft soil sites depends on the ratio of the fundamental period of the structure to the predominant period of the soft soil site, T/T_S. Similarly, strength-reduction factors for ground motions recorded on soft-soil sites exhibit strong variations with changes in the T/T_S ratio. For periods closer than the predominant period of the site (i.e., $T/T_S \approx 1$) R_μ is much larger than the target ductility. For systems with periods shorter than two thirds of the predominant period of the soil site, the strength-reduction factor is smaller than the target ductility, whereas for systems with periods longer than two times T_S the strength-reduction factor is approximately equal to the target ductility. Further discussion on the strength reduction due to nonlinear behavior in structures on soft soil sites as well as simplified expressions can be found in Miranda (1993, 1996).

The dispersion on strength-reduction factors have been recently studied (Miranda, 1993; Riddell, 1995). These studies have concluded that with the exception of very short periods ($T < 0.2$ s), the coefficient of variation (COV) of R_μ is approximately period independent and that the dispersion increases with increasing displacement ductility ratio. COV's vary from 0.2 for ductility ratios of 2 to 0.5 for ductility ratios of 6.

Nassar and Krawinkler (1991) and Miranda (1993) studied the influence of earthquake magnitude and epicentral distance on the strength-reduction factors. Both studies concluded that the effect of both parameters is negligible on R_μ.

FIGURE 2.
RESULTS OF STATISTICAL STUDIES COMPARED WITH THE PROPOSED EQUATION.



Miranda (1996) has shown that the use of approximate reduction factors like those computed with equation 2 combined with the use of smoothed linear elastic response spectra (SLERS) can lead to very good estimates inelastic strength demands (i.e., lateral strength required to control displacement ductility demands).

MODIFICATIONS FOR MDOF SYSTEMS

The R_μ factors previously discussed can be used for the design of structures which can be approximately modeled like a SDOF system. However, most structures need to be modeled as multi-degree-of-freedom (MDOF) systems and have a much more complex behavior than SDOF systems, particularly in the nonlinear range. Thus, the R_μ factors for SDOF systems need to be modified for the design of MDOF structures. It is proposed that the R_μ factor be multiplied by a R_M modifying factor that takes into account the possible concentration of displacement ductility demands in certain floors, thus the product of R_μ and R_M represents the maximum strength reduction factor that will produce an adequate control of story displacement ductility demands in structures that have a strength equal to the design strength. The R_M factor is defined as follows

$$R_M = \frac{R_{MDOF}}{R_{SDOF}} = \frac{R_{MDOF}}{R_\mu} = \frac{F_{y\,SDOF}}{F_{y\,MDOF}} \tag{3}$$

where R_{MDOF} is the ratio of the lateral yielding strength required in the MDOF structure to remain elastic to $F_{y\,MDOF}$ which is the lateral yielding strength required in the MDOF structure to avoid story displacement ductility demands larger than the maximum tolerable story displacement ductility ratio μ_i; and R_{SDOF} is equal to the previously defined R_μ factor.

A study of the R_M factor is currently being conducted by the author. As part of this study, three reinforced-concrete SMRSF 8, 12 and 16 stories high were designed according to a strong column-weak beam philosophy and were subjected to three ground motions with a variable amplitude until maximum story displacement ductilities of 3, 4 and 5 were produced and until the buildings remain totally elastic. Strength reduction factors for equivalent SDOF models of the buildings undergoing the same levels of displacement ductility demands when subjected to the same records were also computed. The equivalent SDOF systems had a period of vibration equal to the fundamental period of vibration of the MDOF structures. **Table 1** shows the R_M factor computed for story displacement ductility ratios of 3, 4 and 5 when subjected to the three ground motions. Although these results are only preliminary, two general trends can be observed: (a) R_M decreases with increasing story displacement ductility ratio (design base shear in MDOF structures increases with respect to SDOF structures with increasing ductility ratio); (b) R_M decreases with increasing number of stories (design base shear in MDOF structures increases with respect to SDOF structures with increasing number of stories).

Based on these results and other limited results presented by Nassar and Krawinkler (1991), the following preliminary equation is proposed for R_M

$$\mathrm{R}_M = \left[1 + 0.15\,\mathrm{T}^2 \cdot \mathrm{Ln}(\mu)\right]^{-1} \tag{4}$$

TABLE 1.
MODIFICATION OF STRENGTH REDUCTION FACTORS FOR MDOF STRUCTURES

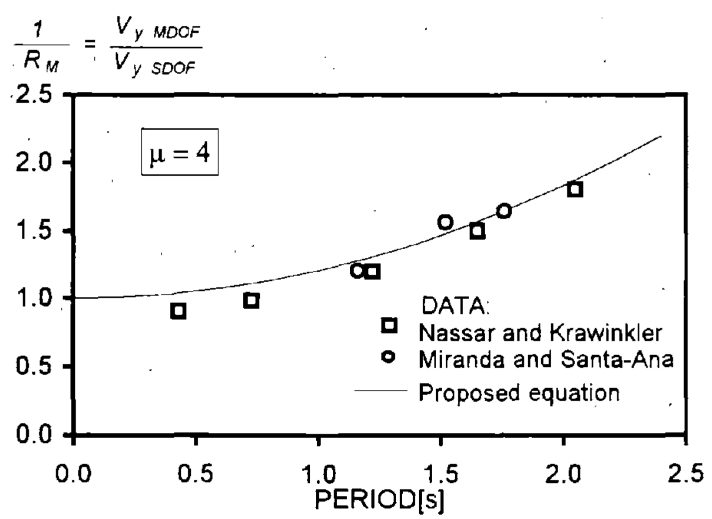
NUMBER OF STORIES	PERIOD OF VIBRATION	STORY DUCTILITY	R_M SCT19.EW	R_M 0625.EW	R_M 5625.NS	$\overline{R}_M$
8	1.16 s	3	0.85	0.83	0.88	0.85
		4	0.84	0.80	0.85	0.83
		5	0.82	0.79	0.84	0.82
12	1.52 s	3	0.67	0.71	0.78	0.72
		4	0.56	0.68	0.68	0.64
		5	0.49	0.60	0.60	0.56
16	1.76 s	3	0.61	0.72	0.67	0.67
		4	0.61	0.61	0.62	0.61
		5	0.60	0.60	0.64	0.61

where T and μ are the period of vibration and the maximum tolerable story displacement ductility demand in the MDOF structure, respectively. A comparison of available data with the results of Eq. 4 is shown in **Figure 3**. Caution should be exercised in the use of Eq. 4 for design purposes as it is based on only few results. Furthermore, it is intended only for regular buildings in plan and in elevation and designed with strong columns-weak beams, so the use of Eq. 4 for other situations can lead to unconservative results.

REDUCTIONS DUE TO STRUCTURAL OVERSTRENGTH

For design purposes $R_\mu \cdot R_M$ corresponds to the maximum reduction in strength that can be used in order to limit the maximum story displacement ductility demand to a maximum tolerable limit the pre-determined target ductility μ_i in a structure that will have a lateral strength equal to

FIGURE 3.
MODIFICATION FOR STRENGTH REDUCTION FACTORS FOR MDOF STRUCTURES



the design lateral strength. An additional strength reduction can be considered in the design of a structure to take into account the fact that structures usually have a lateral strength higher than the design strength. These additional reductions can be divided into reductions due to *element* overstrength R_{SE} which accounts for the increase the lateral strength of the structure from the design strength to the strength associated to the formation of the first plastic hinge and reductions due to redundancy, strain hardening and other factors R_{SS} which increase the lateral strength of the *structure* from the strength associated to the formation of the first plastic hinge to the strength associated to the formation of a mechanism. Thus the suggested reduction factor to be used in design would be given by:

$$R = R_{\mu} \cdot R_M \cdot R_{SE} \cdot R_{SS} \tag{5}$$

For a more detailed discussion on strength reductions due to overstrength the reader is referred to Miranda (1991) or ATC (1995a).

IMPLEMENTATION OF *R* FACTORS IN PERFORMANCE-BASED DESIGN

In performance-based design an adequate design is produced when a structure is dimensioned and detailed in such a way that the local deformation demands are smaller than their corresponding maximum tolerable limits for each performance level. Ideally, the deformation demands and deformation capacities must be checked at the critical region of all members (i.e., at all plastic hinges) by checking the maximum strain, the maximum strain ductility ratio μ_{ε}, the maximum curvature, the maximum curvature ductility ratio μ_{ϕ}, the maximum rotation or the maximum rotation ductility μ_{θ}, with their corresponding limits, however in the preliminary design of a structure the final sizing and detailing is not known, and other parameters at a more global level are more suitable. For preliminary design purposes the author believes that, with the information known to date, the best parameters to achieve an implementation of performance-based design are the story displacement ductility demand and the interstory drift demand, which are related to each other by the story yield displacement. While these parameters do not take into account for cumulative damage in structural members and may have other disadvantages, they have several important advantages: (a) are very simple parameters; (b) structural engineers are familiar with them; (c) most experimental research is based in these parameters, so with a careful calibration in the maximum tolerable limits they can provide an adequate damage control for different performance levels.

The limits in story ductility demands, as well as the limits in interstory drift, vary with the structural system and with the performance level. For example, the maximum tolerable story ductility demand in a steel special-moment-resisting-space frame (SMRSF) is larger than for a concentrically-braced steel frame. Similarly, for the steel SMRSF the maximum tolerable demands will be different for example in the *Life Safe performance level* and for the *Near Collapse performance level*. Thus, during the preliminary design of a structure there is a need to estimate the lateral strength (lateral load capacity) of the structure that is required in order to limit the global (structure) displacement ductility demand and the global drift demand to a certain limit which results in the adequate control of local (i.e. story) ductility demands and interstory drifts. If the elastic design spectra are known for each *earthquake design level*, the R factors permit an estimation of such required lateral strength, particularly for the life safe, near collapse and collapse performance levels. Implementation of R factors in performance-based design requires the specification of such maximum tolerable story ductility demands and maximum tolerable interstory drift demands for each structural system and for each performance level. An important contribution to presently proposed performance-based design methodologies would be the specification and calibration of such limits.

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Energy Concepts and Damage Indices

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SUMMARY

The inadequate behavior, during recent seismic events, of buildings designed according to current earthquake-resistant codes, has given place to intense discussions regarding the need to revise these codes, and the way in which earthquake-resistant design is currently conceived (i.e., current design methodologies).

One concept that can help in formulating a rational and transparent earthquake-resistant design approach is that of performance-based design. In performance-based design, the desired behavior of the building during ground motions of different intensities (design objectives), should be established in a qualitative manner, before the numerical phase of the overall design process starts. Then, the qualitative definition of these design objectives should be quantified, and used as instrumental design information during the numerical phase.

One way in which the design objectives can be introduced into the overall design process is to establish, during the numerical phase, a design methodology that incorporates energy concepts and damage indices. Although currently it is possible to formulate performance-based numerical design methodologies, it is not possible to implement them in practice. It is still necessary to carry out ambitious research programs that can help us understand some of the many issues that currently prevent us, from achieving such implementation.

PERFORMANCE-BASED EARTHQUAKE-RESISTANT DESIGN

One issue that needs to be clearly understood, before formulating an earthquake-resistant design procedure, is that the overall design process should include all those activities that span from the conception of an efficient solution to the design problem at hand, to those that can make possible an adequate quality control during the fabrication (construction), maintenance and operation of the building. Recently, the Vision 2000 Committee of SEAOC (1995), has envisioned a performance-based overall design process, that consists in three phases as follows:

- **Conceptual Phase.** This phase should focus on the conception of an efficient solution to the design problem at hand. To do so, it is first necessary to define the design objectives, which is the set of the desired behavior(s) of the building, generally formulated in terms of acceptable structural and nonstructural damage, for all relevant levels of design ground motion. Next, it is necessary to establish, according to the local seismicity and the design objectives, if the construction site is suitable. If the site is suitable, the designer proceeds to the conceptual design, in which the designer conceptually establishes the global and structural configuration of the building, the structural systems and materials, and the foundation and nonstructural systems. It is suggested that the postelastic behavior of the building is contemplated at this stage of the design; that is, the conceptual design should demand from the designer a clear understanding of the desired nonlinear response of the building. The intuition, experience and good judgement of the designer are essential to a successful formulation of the Conceptual Phase of the overall design process. The Numerical Phase *should not* be started unless the Conceptual Phase has been concluded satisfactorily.

- **Numerical Phase.** Once the Conceptual Phase is finished, the overall design process proceeds to the Numerical Phase, which is constituted by two steps: Preliminary Design and Final Design. These two steps involve the sizing and detailing of the structural and nonstructural systems. Any numerical methodology used to carry out the Numerical Phase should be: *transparent*, so that the designer can lay out numerically a solution to the conceptual formulation of the design problem; *flexible*, so that the designer can solve the Numerical Phase for different design objectives and structural systems; *simple*, to make possible its practical application; and *concise*, so that the designer can consider improving or reformulating its initial conceptual design during the preliminary stages of the Numerical Phase. One of the largest challenges of performance-based earthquake engineering is to formulate and develop a numerical methodology that has the above mentioned characteristics.

- **Implementation phase.** During this phase, the quality of the design should be guaranteed by means of a detailed and independent revision of the same. Also, this phase should contemplate adequate quality control during the fabrication (construction) of the building, and of its maintenance, occupation and function.

A PERFORMANCE-BASED NUMERICAL PHASE

In this section, the primary focus will be on the design of reinforced concrete (RC) ductile frames. Nevertheless, the general ideas can be applied to the design of buildings having other structural systems and/or materials. Traditionally, earthquake-resistant design has been layed out as a demand-supply problem. First, all relevant seismic demands in the building have to be estimated, and then they must be satisfied with adequate seismic supplies as follows:

$$\begin{array}{ccc} \textit{SEISMIC DEMANDS} & \leq & \textit{SEISMIC SUPPLIES} \\ \textit{of} & & \textit{of} \\ \textit{Stiffness} & & \textit{Stiffness} \\ \textit{Strength} & & \textit{Strength} \\ \textit{Maximum and cumulative} & & \textit{Maximum and cumulative} \\ \textit{deformation capacity} & & \textit{deformation capacity} \end{array} \qquad \textbf{(1)}$$

All relevant aspects of Equation (1) should be satisfied efficiently, from technical and economical point of views, under the restrictions imposed by the design objectives. Ideally, Equation (1) should be formulated explicitly for all the levels of design ground motion under consideration.

USE OF DAMAGE INDEXES

Damage control in a building is a complex task, and there are several response parameters that can be instrumental in determining the level of damage that a particular structure suffers during a ground motion. Among these response parameters, the following seismic demands are relevant: deformation (characterized either by the interstory drift index or by the maximum and cumulative displacement ductility ratios), relative velocity, absolute acceleration, plastic energy dissipation and viscous (or hysteretic) damping energy dissipation. It follows that one way of controlling the level of damage in a structure consists in controlling its maximum response. This implies the need to establish limits to the maximum or cumulative demands of some or all of the above response parameters, and to supply the structure with mechanical characteristics that help control its response within the established limits.

Damage indices, that establish analytical relationships between the maximum and/or cumulative response of structural and nonstructural components and the level of damage they exhibit, have been developed and quantified in recent years. A performance-based numerical methodology is possible if, through the use of damage indices, limits can be established to the maximum and cumulative response of the structure, as a function of the desired behavior(s) of the building for the different levels of design ground motion. Once the response limits have been established, it is then possible to estimate the mechanical characteristics that need to be supplied to the building so that its response is likely to remain within these limits.

For instance, the level of damage in typical nonstructural members strongly depends on the interstory drift index (IDI = interstory drift normalized by story height) to which they are subjected to. Through the use of a response index that relates the IDI demands in these members to their level of damage, it is possible to set, according to the acceptable levels of nonstructural damage established by the different limit states, maximum acceptable values for the IDI demands (IDI_{max}) during the different levels of design ground motion. Only then it becomes possible to estimate the mechanical characteristics (usually stiffness and strength), that need to be supplied to the building, so that its maximum global displacement demand, during all design ground motions, is limited to values that are consistent with limiting its maximum IDI demands within the values of IDI_{max} established for the different limit states.

The response index most often used to estimate damage in RC ductile members is that developed by Park y Ang (Park et al. 1987). If some regularity conditions are met, a global value of this index can be used to characterize damage in the ductile members of RC frames (Teran-Gilmore 1996). In this case, the Park and Ang damage index, DMI_{PA}, for a framed structure can be estimated using the following expression:

$$DMI_{PA} = \frac{\delta}{\delta_u} + \beta \frac{E_{H\mu}}{F_y \delta_u} \qquad (2)$$

where δ is the maximum lateral displacement demand during the design ground motion, δ_u the ultimate displacement under monotonically increasing lateral deformation, $E_{H\mu}$ the plastic energy dissipation demand during the design ground motion, F_y the yield strength and β a parameter determined experimentally. A value of DMI_{PA} less or equal than 0.4 can be interpreted as repairable damage; from 0.4 to less than 1.0, as nonrepairable damage; and larger than 1.0 as failure; while 0.15 corresponds to the median of the values of β obtained experimentally. According to the levels of structural damage implied by the limit states established for all levels of design ground motion, it is possible to determine a maximum value of DMI_{PA} for each limit state. Then it becomes possible to estimate the mechanical characteristics (usually stiffness, strength and maximum and cumulative deformation capacities), that need to be supplied to the building, so that its maximum and cumulative deformation demands, during the design ground motions, are limited to values that are consistent with limiting the structural damage to the levels implied by the selected maximum values of DMI_{PA}.

ENERGY AS A DESIGN PARAMETER FOR DUCTILE STRUCTURES

The total plastic energy dissipation capacity of RC ductile members strongly depends on their loading history; and their final failure mode, on the initial pattern of cracks and damage that these members suffer at low levels of deformation. Thus, to assess the level of damage on a RC member after a ground motion, it is necessary to estimate its time-history of load-deformation during this motion, and to compare it to experimental results obtained on similar members subjected to similar time-histories of deformation. Nevertheless, the exact behavior of a member, when the structure is subjected to a particular ground motion, is not as important for design purposes as is the estimation of its expected demands when the structure is subjected to a set of ground motions representative of the design ground motion. That is, what is needed is a general idea of the *expected response* of this member to the design ground motion.

If the average plastic energy dissipation can be used to characterize the expected cumulative nonlinear response of the ductile members of a structure, its use in earthquake-resistant design is possible through the use of simple damage indices, like the one summarized in Equation (2). That is, the designer would be able to establish the mechanical characteristics that need to be supplied to the structure, so that the damage produced by the plastic energy dissipation is consistent with the acceptable level of structural damage. Thus, it becomes necessary to determine whether the way in which the plastic energy is dissipated in the member is instrumental or not in determining its level of damage.

Figure 1 shows constant damage strength spectra obtained from elasto-perfectly-plastic single-degree-of-freedom (SDOF) systems, having equivalent damping coefficients (ξ) of 0.05 and 0.20, and subjected to El Centro N-S ground motion. The strengths summarized in Figure 1 represent those required by the SDOF systems so that their level of damage, after subjected to El Centro N-S motion, is representative of incipient collapse. Two sets of constant damage spectra are shown: one obtained using a response index that neglects the way in which the plastic energy is dissipated (DMI_{PA}), and the other obtained using a response index that heuristically accounts for this energy dissipation (DMI_{MH}). The details on how the spectra shown have been estimated can be found in Teran-Gilmore (1996).

As shown in Figure 1, both damage indices yield similar constant damage strength spectra for ξ of 0.05 and 0.20. This fact implies that the way in which the plastic energy is dissipated has a small influence in the final state of the SDOF systems or, in other words, that the plastic energy dissipated by the SDOF systems is enough to characterize, on average, their cumulative nonlinear behavior. Figure 2 shows similar constant damage strength spectra obtained for the SCT E-W ground motion. The latter motion, recorded during the 1985 Mexico Earthquakes, has a very narrow frequency content around a fundamental period of excitation of 2 sec. As shown in Figure 2, the strength spectra derived from DMI_{MH} has larger ordinates than those derived from DMI_{PA} for SDOF systems having a period around the fundamental period of excitation.

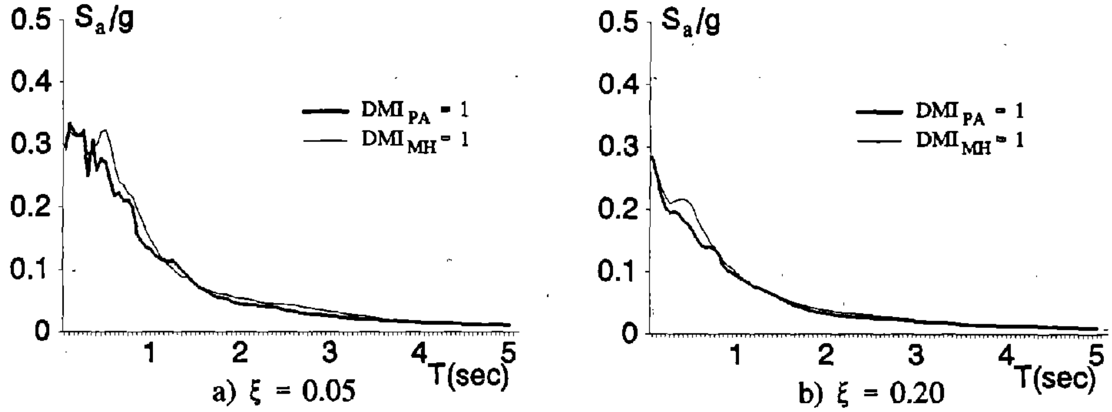


Figure 1 Constant damage strength spectra, El Centro N-S

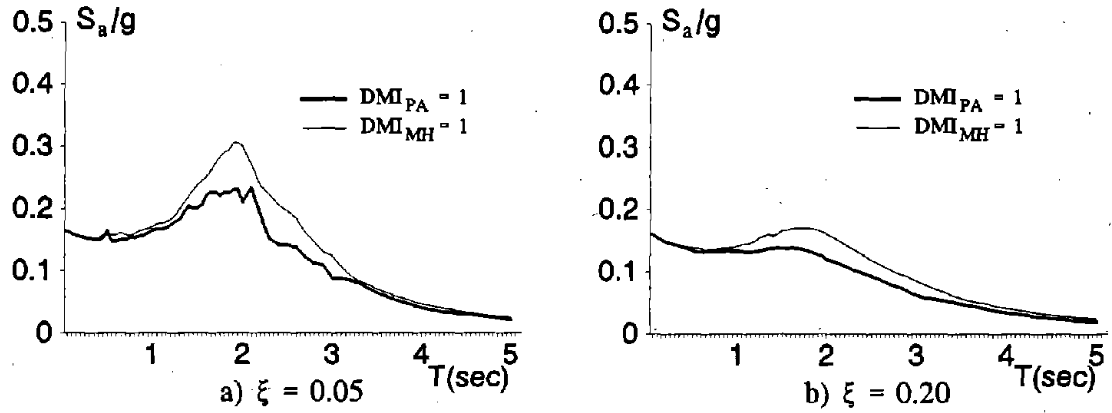


Figure 2 Constant damage strength spectra, SCT E-W

The results shown in Figure 1, illustrate the conclusion derived from an extensive statistical study of the response of SDOF systems to ground motions having different durations and frequency content: due to the random nature of ground motion, the response of the SDOF systems does not tend to concentrate in load cycles of a given amplitude, but rather the response will oscillate between cycles of different amplitude. Thus, the plastic energy is not dissipated in a specific manner, and the total plastic energy demand during the ground motion is, on average, a good way of characterizing the cumulative hysteretic response of the earthquake-resisting system. Furthermore, the total plastic energy demand may be used, in a reasonable manner, in lieu of the time-history of the plastic energy dissipation to estimate failure due to low-cycle fatigue. As shown in Figure 2, an exception to the above is the response of SDOF systems subjected to ground motions with a very narrow frequency content, particularly when the period of these systems is close to that of the excitation.

The above conclusions can not be extended to other levels of damage significantly different to incipient collapse. The statistical studies from which the above conclusions were derived only considered incipient collapse. This issue needs to be clarified through further research.

A METHODOLOGY FOR THE NUMERICAL PHASE

Although current earthquake-resistant design procedures can not be used to lay out adequately all aspects of Equation (2), any procedure formulated as an alternative to current ones should benefit from the knowledge and experience of practical designers. Some of the changes, that are urgently needed to improve earthquake-resistant design, are to provide the Conceptual and Implementation Phases with the place and importance they deserve within the overall design process, and to actualize our numerical design methodologies.

For many years, methods of elastic analysis have provided the designer with a reasonable tool to estimate the local strength "demands" in earthquake-resistant structures. Thus, it seems convenient to keep the use of elastic methods of analysis for this purpose. Nevertheless, the sizing of the structural elements and the determination of other seismic supplies should not be based on the use of such methods. Previous to carrying out an elastic analysis, the mechanical characteristics of the structure, other than strength, should be determined using a numerical procedure that involves the use of damage indices and energy concepts.

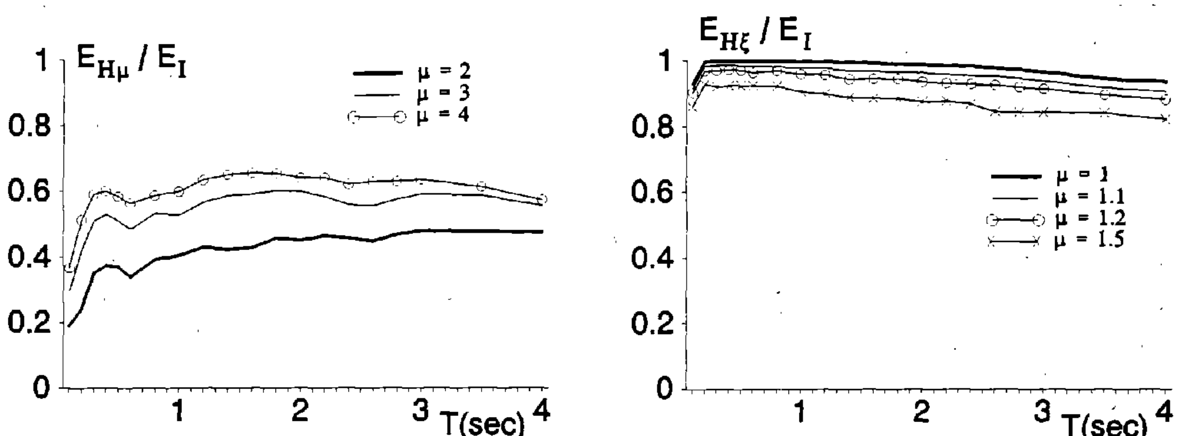
The first thing the designer needs to accomplish during the Numerical Phase, is the quantification, through the use of damage indices, of the qualitative definition of the desired behavior(s) of the building for the different levels of design ground motion. This quantification implies establishing limits to the maximum or cumulative demands of all relevant response parameters. Then, the designer needs to establish the mechanical characteristics, other than strength, that should be supplied to the structure, so that its response is controlled within the specified limits. These mechanical characteristics will be denoted as design parameters and, for simplicity sake, it is proposed that they be estimated using a SDOF model of the structure and the maximum demands (through the use of response design spectra) of all relevant response parameters. That is, the designer should establish the mechanical characteristics that should be supplied to the SDOF model so that its response satisfies the response limits. The mechanical properties estimated in the SDOF model constitute the design parameters for which the structure should be designed. The author has discussed in detail the definition and use of SDOF models to estimate the response of multi-degree-of-freedom structures (Teran-Gilmore 1996).

Some of the most common design parameters are: the maximum fundamental period of translation, which defines the minimum stiffness supply in the structure; the viscous damping coefficient, which can be used for the design of passive energy dissipation systems; the global maximum and cumulative deformation capacities of the structure, which set requirements for the detailing of the structural members and their supports and connections; and the maximum global displacement ductility demand that the structure may undergo during the design ground motion, which sets requirements for the lateral strength that need to be supplied to the structure. It is important to emphasize that it is necessary to estimate design parameters for all relevant levels of ground motion and their corresponding limit states.

Once the critical set of design parameters have been established for all relevant limit states, the designer can size the structural members (stiffness design), estimate their longitudinal reinforcement (strength design), and estimate their transverse reinforcement and detailing (deformability design). It should be emphasized that, by using a methodology as the one discussed above, the designer can obtain a reasonable idea of which limit state controls the design before the structural members are sized and the elastic analysis carried out. In this way, appropriate decisions can be taken in the preliminary stages of the design procedure.

To make all the above possible, the designer should handle simultaneously, during the Numerical Phase, the maximum demands of all relevant response parameters (e.g., displacement, velocity, acceleration, and plastic and viscous damping energy dissipation). The explicit definition of design spectra for all the above mentioned response parameters, may result too cumbersome for practical application. Recently, the author has studied the relationships that exist between the energy inputed by the ground motion to the structure (input energy, E_I) and the plastic energy ($E_{H\mu}$) and viscous damping energy ($E_{H\xi}$) dissipated by the same; and between all these energy demands and other relevant seismic demands. From this experience, the author has learned that these relationships are stable and can be quantified through simple expressions.

To illustrate the nature of these expressions, Figure 3a shows the average $E_{H\mu}$ to E_I ratio, computed on SDOF systems having elasto-perfectly-plastic behavior and a ξ of 0.05, and subjected to 20 synthetic ground motions with a frequency content and strong motion duration similar to El Centro N-S. As shown, this ratio is practically independent of the period (T) of the SDOF systems and fairly independent of the displacement ductility ratio (μ) for $\mu \geq 3$. If the $E_{H\mu}$ to E_I ratio can be estimated through simple analytical expressions (which according to the experience of the author is possible), it is conceivable to estimate, during the Numerical Phase, the $E_{H\mu}$ demands in the structure using a design E_I spectra and the above mentioned ratio. Similar conclusions can be derived, from Figure 3b, about the stability and possible use of the $E_{H\xi}$ to E_I ratio during the Numerical Phase. Figure 3b was obtained from SDOF systems having a ξ of 0.20 and subjected to the same 20 ground motions.



a) Plastic energy to input energy, ξ = 0.05   b) Viscous energy to input energy, ξ = 0.20

Figure 3 Energy ratios for synthetic ground motions having duration and frequency content similar to El Centro N-S

Although not illustrated in this paper, there also exist stable and relatively simple relationships between the above energy demands and the maximum demands of displacement, relative velocity and absolute acceleration. As stated above, it is conceivable to use a design E_I spectra to estimate or account for these demands during the Numerical Phase. Thus, it is worthwhile to study, in more detail, the above relationships and their possible use in formulating a flexible, transparent, simple and concise numerical methodology that helps the designer establish the design parameters of the earthquake-resistant structure at hand.

Once the design parameters have been established, the designer can proceed to size the structural members and carry out an elastic analysis to estimate their strength demands. The elastic analysis should be carried out using strength design spectra, obtained according to the design parameters, representative of all the limit states under consideration. The designer can then proceed to the preliminary detailing of the structure, which consists in the determination of the longitudinal and transverse reinforcement, and its detailing. Finally, it is necessary for the designer to establish if the design is satisfactory or not. One of the most critical aspects of the revision of the design is to determine if the actual maximum and cumulative deformation capacities of the structure are consistent with those established as design parameters. For this purpose, the nonlinear pushover and time-history analyses of the structure are useful. It is important to note that currently, not enough information and analytical tools are available to estimate, in a reliable manner, the real deformation capacity of RC members or to carry out nonlinear analysis.

CONCLUSIONS AND FINAL COMMENTARIES

To establish a rational performance-based design procedure, it becomes necessary to define and incorporate, in an explicit manner, the design objectives into the overall design process. For this purpose, the use of damage indices and energy concepts is useful. Any earthquake-resistant design procedure, proposed as an alternative to current ones, should have a general format, in such a way that the implementation details of the procedure can be actualized, as our understanding of the seismic phenomena improves, without alterations to its basic format.

The exact behavior of a structure to a particular ground motion is not as important, for design purposes, as the estimation of its average response to a set of ground motions that are representative of the design ground motion. Thus, predicting the exact response of a structure is not essential to its performance-based design. Actually, it is not possible to implement in practice performance-based design procedures. It is still necessary to undertake ambitious experimental, analytical and field research programs.

Finally, the reader is encouraged to seek out other publications where the problems of current earthquake-resistant design procedures and the implementation of a performance-based numerical methodology are discussed in detail (Bertero y Bertero 1992, Teran-Gilmore 1996). The use of a performance-based numerical methodology is illustrated, in these publications, for the design of RC buildings with 2, 10 and 30 stories.

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Analysis of Buildings Incorporating Supplemental Dampers

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SUMMARY

Passive seismic supplemental dampers are being used for new and retrofit building construction in California. The primary goal to date of the engineers implementing such damping hardware is *damage control*. This goal is achieved by using the dampers to dissipate earthquake-induced energy, and thereby limit the deformations in the remainder of the seismic framing system.

The widespread implementation of passive seismic dampers has been hampered by the lack of guidelines and commentary for design professionals and building officials. Such guidelines and commentary are included in the *Guidelines for the Seismic Rehabilitation of Buildings* (FEMA, 1997), a resource document (FEMA 273) being developed in the United States with funding from the Federal Emergency Management Agency. The guidelines presented in FEMA 273 for the analysis and design of buildings incorporating passive supplemental dampers were developed by the authors, and represent the state-of-knowledge in the field of supplemental damping at the time of this writing. This paper describes the means by which supplemental damping (energy dissipation) hardware is classified for analysis purposes, and outlines modeling and analysis procedures developed for implementing supplemental dampers.

INTRODUCTION

Seismic framing systems must be capable of absorbing and dissipating energy in a non-degrading manner for many cycles of substantial deformation. In modern conventional construction, energy dissipation occurs in plastic hinge zones in members of the structural frame that routinely form part of the gravity load resisting system. Such energy dissipation is accompanied by substantial nonlinear response (characterized by ductility) and constitutes damage to the seismic framing system. As evinced by experiences following the 1989 Loma Prieta and 1994 Northridge earthquakes, structural damage is often difficult and expensive to repair following an earthquake.

The economic losses resulting from the Loma Prieta and Northridge earthquakes has prompted the earthquake engineering community to embrace the concept of performance based earthquake engineering. Although the basic objective of performance based earthquake engineering is to produce structures that respond in a more reliable manner during earthquake shaking, many engineers associate performance based earthquake engineering with enhanced performance (i.e., damage control). Supplemental dampers can deliver enhanced performance with respect to conventional framing systems.

The objective of adding damping hardware to new and existing construction is to dissipate much of the earthquake-induced energy in disposable elements not forming part of the gravity framing system — thus permitting easy, and relatively inexpensive, replacement (if necessary) of the hardware following an earthquake. Much experimental research has been conducted on the use of supplemental damping hardware for new and retrofit seismic construction. This research is not described in the paper. The reader is referred to Aiken (1990, 1993), ATC (1993), Constantinou (1993), Reinhorn (1995), and Whittaker (1989) for detailed information.

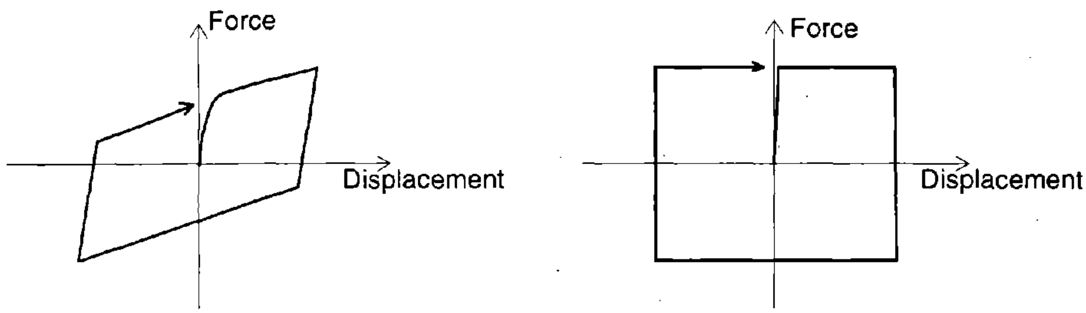
The widespread application of supplemental dampers has been inhibited by the lack of comprehensive analysis, design, and testing guidelines for design professionals and building officials. The scope of this subject is broad, and the length of this paper is limited. As such, only guidelines for the classification and analysis of supplemental dampers are described below. For much additional information, the reader is referred to FEMA 273 (FEMA, 1997).

CLASSIFICATION OF SUPPLEMENTAL DAMPERS

GENERAL

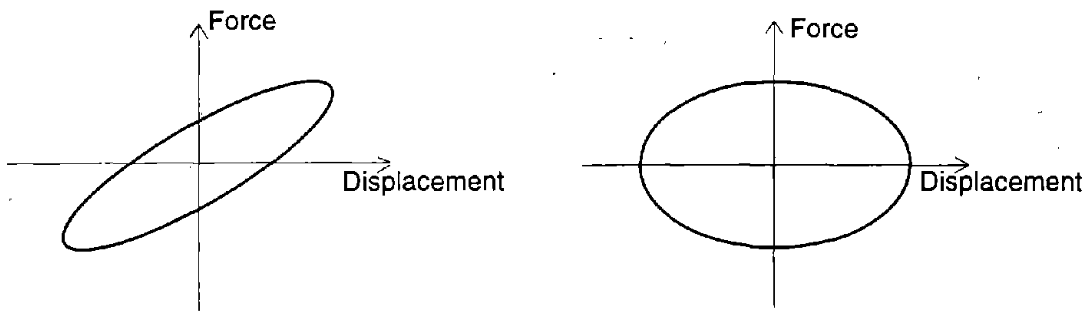
Passive supplemental dampers can be classified as hysteretic, velocity-dependent, or other (FEMA, 1997). Examples of hysteretic systems include devices based on yielding of metal and friction. **Figure 1** shows sample force-displacement loops of hysteretic dampers. Examples of velocity-dependent systems include dampers consisting of viscoelastic solid materials, dampers operating by deformation of viscoelastic fluids (e.g., viscous shear walls), and dampers operating by forcing fluid through an orifice (e.g., viscous fluid dampers). **Figure 2** illustrates the behavior of these velocity-dependent systems. Other systems have characteristics which cannot be classified by one of the basic types depicted in either **Figures 1** or **2**. Examples are dampers made of shape memory alloys, frictional-spring assemblies with recentering capabilities, and fluid restoring force/damping dampers. For information on these dampers, the reader is referred to ATC (1993), EERI (1993), and Soong and Constantinou (1994). Only hysteretic and velocity-dependent dampers are discussed in this paper.

Some types of supplemental damping systems can substantially change the force-displacement response of a building by adding strength and stiffness. Such influence is demonstrated in **Figure 3** for metallic-yielding, friction, and viscoelastic dampers. Note that these figures are schematic only and that the force-displacement relation for the central figure assumes that the framing supporting the friction dampers is rigid. Viscous damping systems will generally not substantially change the force-displacement response of a building.



a. Metallic-yielding damper b. Friction-damper

FIG. 1. FORCE-DISPLACEMENT RELATIONS FOR HYSTERETIC DAMPERS



a. Viscoelastic damper b. Viscous damper

FIG. 2. FORCE-DISPLACEMENT RELATIONS FOR VELOCITY-DEPENDENT DAMPERS

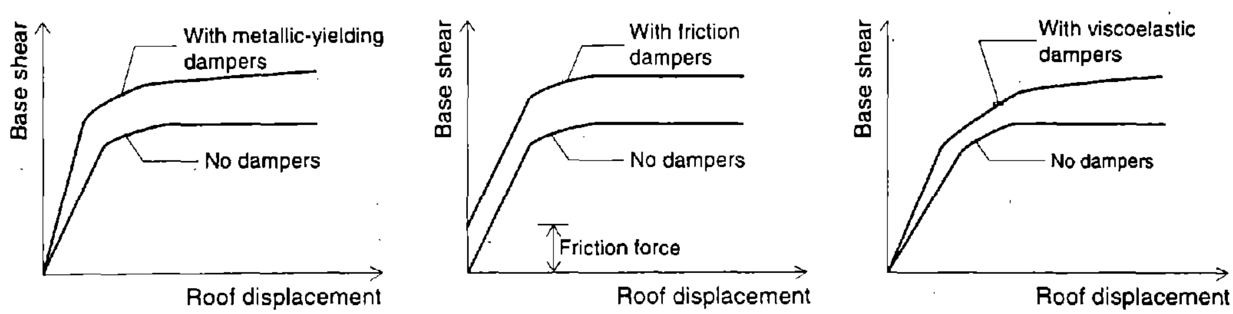


FIG. 3. INFLUENCE OF SELECTED SUPPLEMENTAL DAMPERS ON THE FORCE-DISPLACEMENT RELATION OF A BUILDING

HYSTERETIC DAMPERS

Hysteretic dampers exhibit bilinear or trilinear hysteretic, elasto-plastic or rigid-plastic (frictional) behavior, which can be easily captured with structural analysis software currently in the marketplace. Details on the modeling of metallic-yielding dampers may be found in Whittaker (1989); the steel dampers described by Whittaker exhibit stable force-displacement response and no temperature dependence. Friction devices are described by Aiken (1990) and

Nims (1993); the devices tested by Aiken and Nims responded with box-like hysteresis and no temperature dependence.

VELOCITY-DEPENDENT DAMPERS

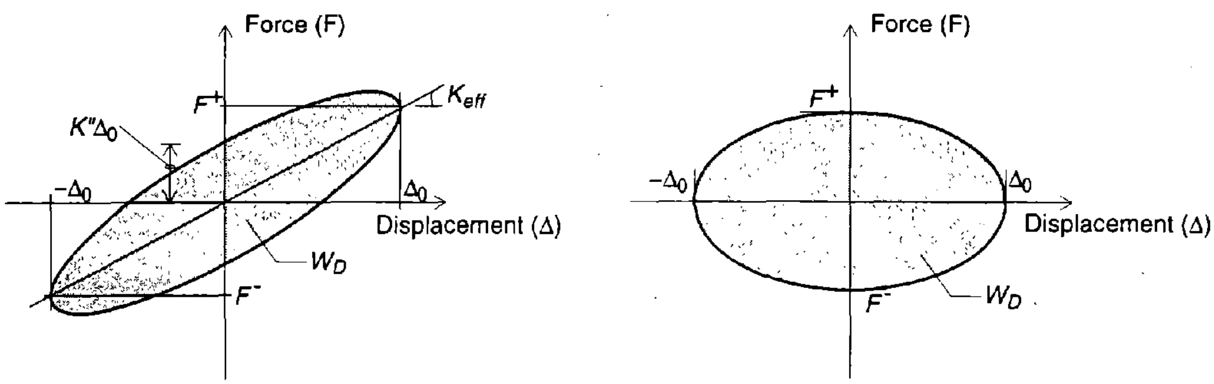
Solid viscoelastic dampers typically consist of constrained layers of viscoelastic polymers. They exhibit viscoelastic solid behavior with mechanical properties dependent on frequency, temperature, and amplitude of motion. A force-displacement loop for a viscoelastic solid device, under sinusoidal motion of amplitude Δ_0 and frequency ω, is shown in **Figure 4a**. The force in the damper may be expressed as:

$$F = K_{eff}\Delta + C\dot{\Delta} \tag{1}$$

where K_{eff} is the effective stiffness (also termed the storage stiffness K') as defined in **Figure 4a**, C is the damping coefficient, and Δ and $\dot{\Delta}$ are the relative displacement and relative velocity between the ends of the damper, respectively. The damping coefficient is calculated as:

$$C = \frac{W_D}{\pi\omega\Delta_0^2} \tag{2}$$

where W_D is the area enclosed within the hysteresis loop and ω is the angular frequency of excitation. The damping coefficient C is also equal to the loss stiffness (K'') divided by ω.



a. Viscoelastic damper

b. Viscous damper

FIG. 4. PARAMETER DEFINITION FOR VELOCITY-DEPENDENT DAMPERS

Parameters K_{eff} and C are dependent on the frequency, temperature, and amplitude of motion. The frequency and temperature dependence of viscoelastic polymers generally vary as a function of the composition of the polymer. The standard linear solid model (a spring in series with a Kelvin model), which can be implemented in commercially-available structural analysis software, is capable of modeling behavior over a small range of frequencies, which will generally be satisfactory for most projects.

Fluid viscoelastic devices, which operate on the principle of deformation (shearing) of viscoelastic fluids (ATC, 1993), have behavior which resembles that of solid viscoelastic devices. However, fluid viscoelastic devices have zero effective stiffness under static loading conditions. Fluid and solid viscoelastic devices are distinguished by the ratio of the loss stiffness to the effective or storage stiffness. This ratio approaches infinity for fluid devices and zero for solid viscoelastic devices as the loading frequency approaches zero. Fluid viscoelastic behavior may be modeled with advanced models of viscoelasticity (Makris, 1993). However, for most practical purposes, the Maxwell model (a spring in series with a dashpot) can be used to model fluid viscoelastic devices.

Pure viscous behavior may be produced by forcing fluid through an orifice (Soong and Constantinou, 1994; Constantinou, 1993). The force output of a viscous damper (**Figure 4b**) has the general form:

$$F = C_0|\dot{\Delta}|^{\alpha}\,\mathrm{sgn}(\dot{\Delta}) \tag{3}$$

where $\dot{\Delta}$ is the velocity, α is an exponent in the range of 0.1 to 2.0, and sgn is the signum function. The simplest form is the linear fluid damper for which the exponent is equal to 1.0. In this paper, discussion on fluid viscous devices is limited to linear fluid dampers; for a detailed treatment of nonlinear fluid viscous dampers, the reader is referred to Soong and Constantinou (1994).

ANALYSIS PROCEDURES

Linear and nonlinear analysis procedures have been developed for the implementation of supplemental damping systems (FEMA, 1997). The use of linear static and linear dynamic procedures is limited to velocity-dependent hardware and linear response in the seismic framing system (exclusive of the dampers). Nonlinear static and dynamic analysis procedures can be used to implement both hysteretic and velocity-dependent dampers in either linear or nonlinear framing systems. Only the linear static and nonlinear static procedures are described below. For information on the linear dynamic procedure, the reader is referred to FEMA 273 (FEMA, 1997). For nonlinear dynamic analysis, the force-deformation response of the dampers is modeled explicitly. The reader is referred to the list of references at the end of this paper, and the literature in general, for information on nonlinear dynamic analysis.

LINEAR STATIC ANALYSIS

The linear static procedure (LSP) set forth in FEMA 273 (FEMA, 1997) is fundamentally different from the linear analysis procedures adopted in current seismic codes and regulations in the United States. The LSP is a first-mode *displacement-oriented* analysis procedure that computes design actions using an equivalent base shear — a base shear force that when applied to a linearly-elastic mathematical model of a building will produce displacements of the magnitude expected in the yielded (nonlinear) building. However, as the LSP can only be used to implement velocity-dependent dampers in elastically responding buildings, the equivalent base shear is equal to the elastic base shear force calculated using the total reactive weight and fundamental period of the building.

Displacements in a building responding in the elastic range can be reduced by velocity-dependent dampers through added stiffness (viscoelastic dampers only) and added damping. The effective stiffness of viscoelastic dampers (calculated at the expected displacements) must be included in the mathematical model of the building; the added stiffness serves to reduce the fundamental period of the building and thus displacements (at the expense of increased accelerations). The effective damping of the building (β_{eff}) is calculated as the sum of the structural damping in the building frame (typically taken to be 0.05) and the added damping provided by the velocity-dependent dampers (β_d). The calculation of β_d is dependent upon the damping coefficient C (which must be determined at the fundamental frequency of the building), the angles of inclination of the dampers to the horizontal, and the first mode properties of the building. The effective damping of the building is then used to reduce the equivalent base shear (based on 5-percent damping), to a so-called modified equivalent base shear through the use of a damping factor (termed B in FEMA 273). Sample values for B are 1.2 (β_{eff} = 0.10), 1.5 (β_{eff} = 0.20), and 1.7 (β_{eff} = 0.30).

Design actions in components of buildings incorporating velocity-dependent dampers must be checked at three stages: maximum drift, maximum velocity, and maximum acceleration. Assuming that the damper force-displacement response presented in **Figure 4a** is representative of the base shear-roof displacement response of a building incorporating velocity-dependent dampers, component actions must be checked at three stages: maximum drift (at displacement $= \Delta_0$ in the figure); maximum velocity (at displacement = 0 in the figure); and maximum acceleration (at the displacement corresponding to the maximum force). Information on the calculation of component actions at the stages of maximum velocity and maximum acceleration can be found in FEMA 273 (FEMA, 1997).

NONLINEAR STATIC ANALYSIS

The nonlinear static procedure (also termed pushover analysis) is a *displacement-based* method of analysis. For such analysis, the building is represented by either a two- or three-dimensional mathematical model. The stiffness of the supplemental dampers must be represented in the model. Lateral loads are applied in a predetermined pattern, and the mathematical model is incrementally pushed to a target displacement. A force-displacement relation for the building is thereby established. Typically, the force variable is base shear and the displacement variable is roof displacement. The target displacement is established in FEMA 273 by either the coefficient method or the capacity spectrum method. Calculation of the target displacement by the coefficient method is based on the assumption that, for periods greater than approximately 0.5 second (for a rock site), displacements are preserved in a mean sense, that is, mean elastic displacements are approximately equal to mean inelastic displacements. (Note that the degree of scatter in the ratio of elastic and inelastic displacements may be substantial, and that this assumption is likely unconservative for buildings with low strength.) The general form of the target displacement (δ_t) equation is:

$$\delta_t = \prod_i C_i S_a \frac{T_e^2}{4\pi^2} \tag{4}$$

where C_i are coefficients to relate expected inelastic displacements to elastic displacements (all greater than 1.0), T_e is the effective fundamental period of the building (an improved measure of the fundamental period of the building up to first significant yielding), and S_a is the 5-percent damped spectral acceleration at period T_e.

The benefit of hysteretic dampers is recognized through the addition of stiffness to the building. The elastic stiffness of hysteretic dampers should be represented in the mathematical model to calculate T_e. The increased stiffness provided by the hysteretic dampers will reduce T_e and generally reduce the target displacement.

Viscoelastic dampers exhibit effective stiffness which is generally dependent on frequency of motion, amplitude of motion, and temperature. Viscoelastic dampers should be modeled for the purpose of nonlinear analysis as either linear or nonlinear springs representing the effective stiffness of the damper at a fixed temperature and frequency. The effective stiffness calculation should be based on an excitation frequency equal to the inverse of the secant period of the building (including the viscoelastic dampers) at the target displacement. The mathematical model of the building must include the stiffness characteristics of the viscoelastic dampers and their supporting framing. The fundamental period of this model should be used to estimate the target displacement from a response spectrum that is modified from the 5-percent spectrum to account for the viscous damping provided by the dampers. Viscous dampers will generally exhibit little stiffness, and reductions in the target displacement due to added stiffness can be ignored. Modification of the target displacement due to added damping is a key step in the analysis process. The first mode damping provided by the velocity-dependent dampers can be estimated using the procedures set forth in FEMA 273 and outlined above. The effective damping of the supplementally damped building is then used to estimate the damping factor B (see above). The target displacement calculated using a 5-percent damped spectrum is then divided by B to calculate a modified target displacement.

Design actions and deformations are checked at one stage for hysteretic dampers (maximum drift), and at three stages for velocity-dependent dampers (maximum drift, maximum velocity, and maximum acceleration). See above and FEMA 273 for additional information.

SUMMARY AND CONCLUSIONS

Guidelines and commentary for the analysis and design of existing buildings incorporating supplemental damping (energy dissipation) devices are presented in FEMA 273. The guidelines and commentary, developed by the authors of this paper, present comprehensive procedures for the implementation of hysteretic and velocity-dependent dampers in existing construction. These procedures are equally valid for new construction. Linear procedures were presented for the implementation of velocity-dependent dampers in seismic framing systems responding in the linear range. Nonlinear static analysis procedures were described for the implementation of hysteretic and velocity-dependent dampers in seismic framing systems responding in both the linear and nonlinear ranges. The reader is referred to FEMA 273 (FEMA, 1997) for much additional information.

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Earthquake-Resistant Design in Chile after the 1985 Earthquake

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SUMMARY

An analysis of the performance of Chilean buildings after the 1985 earthquake and of the way it influenced the subsequent revisions of the Chilean seismic code are presented. The discussion is focussed on the provisions developed in order to obtain adequate seismic performance during severe earthquake events. Problems to achieve this goal through code provisions are discussed, as well as the basic ideas behind the provisions for the design of industrial structures presently under study.

INTRODUCTION

The March 3, 1985 Central Chile earthquake affected densely populated areas with buildings of different characteristics, most of them designed using earthquake-resistant code provisions. This has been perhaps one of the most interesting events in the last twelve years, since it involved a 7.8 Richter magnitude earthquake and a large number of buildings, housing about 3 million people, located at 200 km or less from the epicenter. Many records were obtained, whose characteristics have been reported elsewhere (Cruz et al., 1988; Monge, 1986). From the earthquake-resistant design point of view it is important to summarize the main lessons obtained from the behavior of buildings designed according to the Chilean code current at that time (Instituto Nacional de Normalización, 1972). This code reflected the state-of-the-art of Chilean practice and research thirty years ago; its provisions were quite simple as they prescribed seismic actions associated to a moderate, rather frequent earthquake ground motion and included a limited consideration of the energy absorption and dissipation capacity of the structure. Nevertheless, the use of buildings with highly redundant, shear-wall structural systems having a considerable degree of overstrength, provided an adequate control of ductility demand and of damage and led to an overall satisfactory seismic performance as described below. This paper is focussed on the code changes implemented after the 1985 earthquake in order to maintain the performance of those structures that exhibited good behavior and to improve that of the structures that did not show a satisfactory performance.

SUMMARY OF BUILDING PERFORMANCE

The damage experienced by the buildings has been documented in several reports. The most comprehensive report about the earthquake is perhaps that of the EERI Reconnaissance Team (Wyllie et al., 1986), while the main lessons learned from the structural performance have been summarized by Cruz et al. (1988); a detailed description of the damage exhibited by both reinforced concrete and masonry buildings occupied by low-income families and the reasons that could explain that behavior have been reported by Flores (1993). A summary of this structural performance follows.

The class of medium to high-rise buildings in Chile may be defined as those having a number of stories between 5 and 30, since in 1985 there were no buildings taller than 30-story high. Practically all of them had a reinforced concrete, shear-wall structure designed using the German DIN code and the American ACI code provisions, mixed according to the judgement of the design engineer. Special attention has been given to the behavior of 415 buildings with a number of stories between 5 and 23, located in the coastal city of Viña del Mar at about 50 km from the epicenter, where a peak ground horizontal acceleration of 0.36g was recorded. Out of the 40 buildings with 12 or more stories, only six suffered important structural damage due to significant torsional plan irregularity, to the presence of short columns not anticipated in the design process, and to the existence of damage not repaired after the July 8, 1971 earthquake (Cruz et al., 1988). Additional reasons for the seismic damage experienced by some reinforced concrete buildings in Santiago were the vertical irregularity of the structural layout, the inadequate design, detailing and/or construction of reinforced concrete elements, and the lack of attention to the design and construction of non-structural partitions. Though the number of buildings having a space frame structural system is not significant compared to the number of shear-wall buildings, it is interesting to note that practically no indication of inelastic behavior was observed in them; this is due to the minimum value of base shear (6% of total weight) required by the 1972 Chilean code, which provided the overstrength needed to preclude the development of inelastic deformations.

Most buildings with four or less stories are made of masonry. Confined masonry exhibited good seismic performance; the cases of buildings with structural damage corresponded with those not satisfying the seismic provisions used in Chilean practice, the most common of them being the lack of one or both of the confining columns of the resisting walls. The behavior of reinforced masonry buildings 3 or 4-story high was by far the worst case of structural behavior; the reasons have been explained in detail elsewhere (Cruz et al., 1988), but may be summarized by stating that this is a typical case of interpretation of a foreign code, in this case the Uniform Building Code provisions with conditions of materials and workmanship quite different from those behind that code. Two facts became clear after the 1985 earthquake: that shear strength of Chilean, ungrouted, hollow clay reinforced masonry is low and very much dependent on quality of workmanship, and that ductility and overstrength of reinforced masonry built in Chile prior to 1985 was less than that implied by the design forces specified by the 1972 Chilean seismic code.

The behavior of small buildings, one or two stories high, was generally quite satisfactory, except for those old buildings made of adobe or unreinforced masonry. Most masonry buildings in Chile have more walls than those strictly needed to resist code forces. This overstrength has been very useful to preclude damage in the cases of poor quality construction or poor structural layout, but this is not the case when the number of stories increases. The main problem found in unreinforced masonry was the lack of an horizontal diaphragm or a sufficiently stiff collar girder to improve both the behavior of wall panels for out-of-plane seismic action and the transfer of inertia forces to the walls parallel to the seismic action.

Finally, it is important to note that there were practically no cases where structural damage could be attributed to soil or foundation problems, except for buildings constructed on steep, sandy slopes near the coast where some sliding occurred, or on the top of ridges where topographical amplification of ground motion was evident.

CODE CHANGES AFTER THE 1985 EARTHQUAKE

One year after the 1985 earthquake, the Chilean Government asked the Chilean Institute of National Standards to revise the 1972 Chilean seismic code, prompted in part by the poor behavior exhibited by the reinforced masonry buildings. An important percentage of these buildings had been financed by the Government and had to be repaired at the expense of fiscal budget. The goals of the revision process were: a) to incorporate the state-of-the-art knowledge, and b) to include some earthquake-resistant design facts used in Chilean engineering practice that proved to be important in the satisfactory seismic performance exhibited by the Chilean buildings. The revision process took a number of years until a new version became official in September 1993 (Instituto Nacional de Normalización, 1993). The main modifications introduced in this revision were: a) a seismic zonation of the Chilean territory; b) a new classification of supporting soil types with a detailed description of each of the four types; c) a new design spectra and seismic coefficients based on the records obtained from the 1985 earthquake as well as from previous earthquake events; d) an explicit recognition of the influence of the structural type and structural material on the seismic response, through the use of a response modification factor R; e) a set of limitations for the use of the static analysis procedure; f) new design provisions to include the effects of torsional response; g) the use of CQC formula to estimate maximum response from the modal maximum values; and h) the control of seismic performance through limitations imposed to interstory drift and interstory torsion angle. The following discussion has been limited to modifications d) and h) above, that are essentially aimed at avoiding the bad experiences of the 1985 earthquake, thus hoping for adequate seismic performance in future earthquake events. Details about the other modifications have been reported by Hidalgo (1992) and Hidalgo et al. (1994).

The response modification factor R reduces the elastic spectral values, and is a function of the period of the structure and of a parameter R_0 that takes into account the type of structural system and the structural material. The basic model for R was developed for single degree of freedom systems by comparing the spectral ordinates of linear elastic models and non-linear elasto-plastic systems with a given ductility factor (Hidalgo and Arias, 1990). In the Chilean code, the ductility factor is replaced by $R_0 + 1$, where R_0 is a parameter that depends, not only on the energy dissipation capacity of the structure, but also on overstrength and other factors used in Chilean engineering practice. In fact, the value of the R_0 for reinforced concrete, shear-wall structures was chosen in such a way that buildings having 10 to 12 stories located in the highest seismic zone, be designed with the same strength that would have been required by the old 1972 seismic code, in order to keep the satisfactory performance exhibited by this type of buildings during the 1985 earthquake. On the other hand, the value of R_o for masonry buildings, 3 to 4-story high, was chosen following experimental and analytical studies performed in Chile for this type of buildings, both prior and after the 1985 earthquake; this meant to increase the strength required by the 1972 code by a factor of the order of 2. **Figure 1** shows the R factor included in the 1996 Edition of the Chilean code (Instituto Nacional de Normalización, 1996) for two types of soils and two types of shear-wall structures: R_o=11 is for reinforced concrete and R_o=4 is for confined masonry and the better type of reinforced masonry. **Figure 1** also shows that the influence of soil on the response modification factor R is only reflected through the parameter T_o, which is reasonable for stiff soils only; Riddell (1995) and Miranda (1996) have found improved relations for soft soils. The values of parameter R_o for other types of building structures were decided with reference to the cases indicated above. Nevertheless, there is an obvious penalization of the space frame system, both steel and reinforced concrete, since their R_o value is the same as for shear-wall reinforced concrete structures.

The other modification discussed in this paper is the set of limitations imposed to the deformations of the structure obtained from the elastic analysis prescribed in the code: the interstory drift evaluated at the center of mass of any story shall not exceed 0.002, provision that is mainly oriented to prevent damage in partitions and non-structural elements during severe earthquakes; moreover, the interstory drift at any point of the plan, measured in addition to that at the center of mass, shall not exceed 0.001, provision intended to control the amount of torsion due to a non-symmetrical plan distribution of structural elements or by a plan with low torsional stiffness. The general feeling among the code committee members was that the methods of analysis prescribed in the seismic code are not able to reproduce the actual response during severe earthquakes, considering the desirable degree of simplicity they must have. Consequently, methods have to be kept simple but adequate performance must be guaranteed as much as possible. Control of deformations, even at an elastic response level, is one alternative to achieve this objective. The first provision discussed above has been successfully used in Chile, although it was not explicitly written in the 1972 code. The second provision is an attempt to avoid structures with excessive torsional response and its appropriateness is yet to be assessed.

In spite of the fact that the provision of a minimum base shear has always been in the Chilean seismic code, it is necessary to emphasize its importance to prevent long-period structures from undergoing excessive deformations and damage. This provision controls the design of most of the space frame structures and of the shear-wall structures with 12 to 15 stories, approximately, particularly when they are located on stiff foundation soils. The influence of this provision in the performance of Chilean buildings during the 1985 earthquake has been discussed above.

Though it is not explicitly declared, there is an obvious preference in the 1993 and 1996 Chilean seismic codes for the shear-wall or braced-frame systems, as compared with the space frame systems, preference that is reflected in a number of provisions. The justification of this decision is the overall satisfactory performance exhibited by these structures during the 1985 earthquake, in spite of the fact that design methods and construction procedures used for these buildings were much less sophisticated than those required when the survival of a structure under severe earthquake events depends on the development of ductile inelastic behavior in the structural elements. Collapse of shear-wall systems is very unlikely, but also considerable overstrength and high redundancy are obtained almost automatically. Nevertheless, extrapolation of this successful experience to other earthquake-prone countries may not be easy for a number of reasons, but this fact cannot be used to deny the value of the Chilean experience.

Less than a year after the 1993 Edition of the Chilean seismic code became official, structural engineers presented some problems related with the application of the new provisions when compared with the results obtained with the 1972 code. The main problems were due to the fact that calibration of R_0 values was performed using static analysis results only, and therefore it was not always valid when modal spectral analysis was used because of the changes introduced in the design spectra and in the formula to combine modal maxima to estimate maximum design values. The Code Committee was called again to study these problems, solved them after completing the required studies, but also addressed the problem of establishing the maximum base shear for low-period structures; the maximum base shear was decided on the basis of the elastic design spectra reduced by the response modification factor R shown in **Fig. 1**, but the resulting values were modified considering Chilean practice. **Figure 2** shows the seismic coefficient for the static analysis for the same type of structures and soil conditions included in **Fig. 1**, located in the highest seismic zone of the country; in the static analysis, R=7 is used for shear-wall, reinforced concrete structures, and R=4 is used for confined and the better type of reinforced masonry. The maximum base shear obtained from the static analysis also applies when the modal spectral analysis is used. The 1996 Edition of the Chilean seismic code (Instituto Nacional de Normalización, 1996) became official in December 1996.

The discussion held during the Code Committee meetings revealed other problems related to the proper use of the code provisions. There are cases of engineers that ignore the philosophy and scope of the provisions and use them improperly. For example, the Chilean provisions for shear-wall reinforced concrete structures are based on the behavior of this type of buildings during the 1985 earthquake; if designers begin to use buildings whose structural characteristics are different from those of the buildings that experienced the earthquake, typically use less cross-sectional area of shear walls divided by floor plan area, and the same code provisions, the future seismic performance of these buildings may not be as good as before. And this may very well happen because the lateral strength required by the code provisions does not force the use of the amount of shear walls for which the code provisions were derived. Further studies are needed in order to solve this loop-hole in the code. Another example is the use of the code provisions for cases not explicitly considered in the code, i.e., precast reinforced concrete space frames having a lesser degree of structural redundancy than the traditional cast-in-place frame. It is rather difficult to solve this type of problems that arise from the proper use of the code provisions, unless a permanent updating process is carried out, or more frequent earthquake events teach us how to improve our earthquake-resistant design practices. Nevertheless, the best way to solve these problems may be to develop simple methods to verify the appropriateness of code-designed structures through a simulation of its behavior under a severe earthquake event.

DESIGN OF INDUSTRIAL STRUCTURES

Earthquake-resistant design of industrial structures may not be as important as that of residential, commercial, or office buildings as far as extended loss of human life during severe earthquake events is concerned, but it is quite important due to the economic loss that may eventually occur. Chile also offers a satisfactory seismic experience in this area, not only after the 1985 earthquake, but also after the 8.4 Richter magnitude earthquake that affected an important industrial area in Southern Chile in May 1960. Lateral strength of industrial structures is generally higher that in residential or office buildings because of the larger factor of safety used to design those buildings; the cost of the structure is small compared to that of the industry and the eventual economic loss due to interruption of production may be very large. However. the structural system is at the service of the industrial layout, shear-wall structures are very rare, and vertical or horizontal bracing, when allowed, are the most common way to provide lateral strength or diaphragm action in steel structures. Nevertheless, the use of unbraced frames is very common. These facts mean that structural redundancy and overstrength might be much less than in residential or office buildings. Moreover, in many cases of structures and equipment, the locations where energy may be dissipated during severe earthquake events are quite defined, namely, at their anchorage to the foundations or to the supporting structures. Extreme care must then be used when designing the anchorage systems, anchor bolts or shear keys, in order to have a proper transmission of the earthquake forces to the foundations and to have a minimum interruption of production in case that inelastic deformations require to repair or fix the anchorage system. Other cases, like the design of electrical equipment or piping systems including their supporting structures, have their own characteristics and special design requirements are needed. Presently, a seismic code for the design of industrial construction is under study and should be available by the end of 1998.

CONCLUSIONS AND RECOMMENDATIONS

The main lessons obtained from the behavior of buildings after the 1985 earthquake may be summarized as follows:

1. Shear-wall, reinforced concrete structural systems with a rather high degree of overstrength provide an adequate control of ductility demand and damage during severe earthquake events.

2. Adaptation of foreign codes to local design conditions must be done carefully, as shown by the poor performance of reinforced masonry buildings.

3. Control of lateral deformations, even at an elastic response level, seems to be an effective tool to achieve satisfactory seismic performance.

The Chilean experience shows that is highly desirable that code provisions be supported by the performance of real buildings during severe earthquakes. It is also important for the code provisions to be effective, that users be familiar with the philosophy and limitations behind those provisions.

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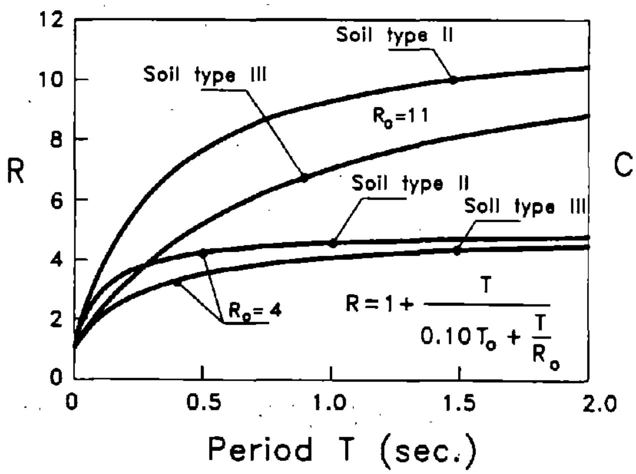

Fig. 1 Response modification factor R.

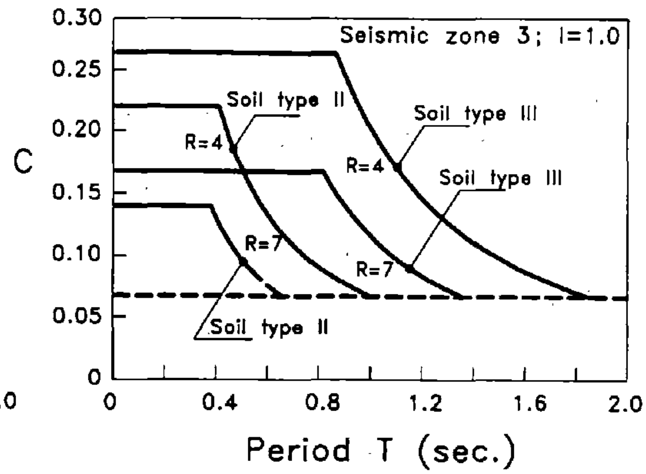

Fig. 2 Seismic coefficient

The 1985 Mexico Earthquake. What We Have Learned Thereafter

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SUMMARY

From the intense research work performed after 1985, some relevant results are presented in the paper. The seismic hazard in Mexico City is characterized by a double amplification of seismic waves due first, to regional geology and then to the soft clay deposits. Totally, spectral ordinates are amplified up to 200 times for some initial frequencies. Structural response is extremely high if the fundamental period of the structure is near to the dominant period of the soil deposits. Nevertheless, ductility is particularly efficient in reducing the response for near-resonance conditions.

The reasons why some structural systems performed better than expected, while other showed a very poor behavior are explained. Some non conventional techniques used for structural rehabilitation of buildings are described, and the trend of the present revision of the seismic codes are briefly commented.

INTRODUCTION

The earthquake of September 19, 1985 has been the most devastating of this century in Mexico. Its magnitude was Ms=8.1, and its epicenter was located in the Pacific Coast, 373 km West of Mexico City. Many lessons have been derived from this event; they are mostly related to the particular situation in Mexico, but many of them can be extrapolated to other countries.

The first lessons were derived from the observation of building performance and from the examination of ground motion records (Rosenblueth and Meli, 1986). Among them: the great amplification of seismic waves in the soft clay deposits of the lake-bed zone, where most of Mexico City is located, the significant variation in amplitude and prevailing periods of seismic waves according to the thickness of those deposits; the resonance effect due to the similarity of fundamental vibration periods of mid-rise buildings and of typical clay layers; the importance of soil-structure interaction; the effects of several common irregularities in the structural scheme, and of the lack of proper detailing, which increased the vulnerability of reinforced concrete structures. Professor Bertero took a particular interest in the initial evaluation understanding the characteristics of this earthquake, particularly regarding the destructive potential of this kind of strong motion in very soft ground (Bertero, 1986).

More than 11 years after that earthquake, a much better understanding of its effects has been achieved, as a result of a great effort of research and of professional work. From the large amount of results from several sources, the author has selected the issues that, from his personal point of view, are the most important for the improvement of the earthquake engineering practice.

SEISMIC HAZARD AND LOCAL EFFECTS

Mexico's seismic hazard is mostly governed by earthquakes produced along the Pacific Coast, where the Cocos Plate subducts the North American Plate, and generates a great number of strong earthquakes: 44 events with magnitudes Ms ≥ 7.0 have been reported in this century. Additionally, some intraplate earthquakes affect other regions of the country.

Seismic intensities produced at epicentral regions by these subduction earthquakes are much smaller than those recorded in other seismic areas. As it can be observed in **Figure 1**, the maximum ground accelerations recorded in the 1985 Mexico Earthquake are about one third of those of the 1985 Chile earthquake, which was also a subduction event and had a smaller magnitude (Ms=7.8). Because of the relatively small accelerations, the damage along the Pacific Coast has been, in general terms, moderate for the magnitude of the earthquakes.

Contrary to the low intensities in epicentral regions, these subduction earthquakes have produced high intensities, and consequently large damage, in some places faraway from the rupture zone. The most striking example at this regard is Mexico City, where the conditions of seismic hazard are very peculiar and deserve special attention.

The attenuation of the amplitude of seismic waves with the distance from the earthquake focus is shown in **Figure 2** for several recent events. As it can be appreciated, amplitudes of high frequency waves in sites of firm soil in Mexico City, follow the general trend of attenuation derived for other sites. On the contrary, low frequency waves in the firm soil sites of Mexico City Valley reach amplitudes of about ten times those corresponding to the general trend. The difference is attributed to characteristics of the deep geological structure of the region (Ordaz and Singh, 1992).

An additional amplification takes place in the so called "lake zone" of the Mexico City Valley, due to the vibration of the thick layers of very soft soil. The main features of the problem can be derived from three Fourier spectra shown in **Figure 3**. One corresponds to the motion recorded in the epicentral region, other at a firm soil site of Mexico City, and at a site in the lake zone of Mexico City. The two sites on firm soil show similar amplitudes for frequencies up to 1 Hz. The site on soft soil in Mexico City shows greater amplitudes for low frequencies than the one on firm soil. For instance, for a frequency of 0.5 Hz, the spectral amplitude in soft soil is about 20 times greater than in firm soil. In conclusion, a double amplification phenomenon takes places in the Mexico City Valley. Low frequency waves are first amplified by the geological structure of the region (about ten times), and additionally by the soft soil layers (up to 20 times more for some critical frequencies). In total, the wave amplitude can be more than 200 times greater than those corresponding to the general trend of attenuation (Singh and Ordaz, 1993).

Another distinct feature of the seismic ground motion in the lake zone of Mexico City is the consistency of the frequency content of the ground motion in a given site. Depending mainly of the thickness of the clay deposits, but also on its consolidation, each site responds with its prevailing frequency, which is independent from the magnitude and specific location of the epicenter in the subduction zone, as shown in **Figure 4**. This last feature has important consequences for earthquake-resistant design of buildings, because a characteristic spectral shape can be assigned to each site.

Dynamic tests have demostrated that the lake-bed clay maintains a linear elastic behavior with low damping ratios, up to shear strains as larger as 1% (Romo, 1995). Therefore, in addition to be capable of extremely large amplifications, the clay layers produce a motion at the surface with constant characteristics and with an amplitude which is proportional to the intensity of the seismic waves incoming from the underlaying firm deposits.

Another peculiarity of the seismic hazard in Mexico City is the great number of seismic sources that can generate significant intensities in the area. The number of moderate intensity events that can occur during the expected life of a building is much greater than in most other areas of high seismic hazard. For that reason, the design for the serviceability limit state is particularly critical. As an evidence of this situation, **Figure 5** shows the return period of maximum ground accelerations in a site of firm soil. The ratio between intensities corresponding to 50 and 475 years return periods is 0.53. The same ratio estimated for a site in Los Angeles is 0.32, according to the EERI Committee on Seismic Risk (1989). The two return periods are close to those commonly associated to design intensities for serviceability and ultimate limit states.

STRUCTURAL RESPONSE TO GROUND MOTIONS IN VERY SOFT SOIL

Typical acceleration response spectra of motions recorded in the lake zone of Mexico City are shown in **Figure 6**. They are characterized by extremely large amplitudes for vibration periods of the system (Ts) equaling the fundamental vibration period of the clay deposits (T_G), which in the lake zone can vary from 1 to 4 seconds, depending on the thickness of the clay layers in each particular site. The shape of the spectra is very similar, and a standarized shape it can be defined in terms of T_S/T_G. Resonance due to similarity between the fundamental period of a structure and that of the underlaying soil is, therefore, a critical problem.

The effect of non linear behavior on the structural response is also very peculiar and can be appreciated from response spectra for perfectly elasto-plastic systems. As shown in **Figure 7**, the peak response is drastically reduced even for rather small ductility factors, $\mu=2$, and completely disappears for $\mu=4$. A generalization of these results is shown in **Figure 8**, where the variation of "ductility reduction factors" is represented in terms of T_S/T_G. Ductility reduction factor, R, is meant as the ratio between the ordinates of the elastic and elasto-plastic spectra for a given ductility and period. For $T_S/T_G=1$, the reduction factor is much greater than μ ; it can be demonstrated that the peak of the reduction factors increases as the band-width of the motion decreases (Ordaz et al., 1993). For $T_S/T_G<1$, R decreases sharply and tends to 1 for $T_S/T_G \rightarrow 0$.

From these results, it can be concluded that the problem of resonance is not as critical as it could seem, from the point of view of safety against structural collapse. Structures possessing a reasonable ductility capacity and with a fundamental period of vibration near to that of the soil, need almost the same strength to withstand the earthquake than those that are far from a resonance condition. On the contrary, from the serviceability point of view, a condition of near-resonance, which involves great amplification of the response, will require incursions in the non-linear range of behavior, even for moderate earthquakes. As these incursions are associated to some level of damage, the proximity to resonance must be avoided, unless a very large safety factor could be achieved.

Considering that, as shown in the previous section, each site has a characteristic dominant period, which remains stable unless significant alterations in soil properties occur, it is possible to define site-specific design spectra. For instance, for a postulated critical earthquake (Magnitude M=8.2 with the nearest possible epicenter to Mexico City) expected response spectra in different sites are shown in **Figure 9**. They have been calculated by attenuation laws and transfer function fitted to the numerous earthquake records available. The great differences in critical period and in maximum spectral ordinate can be easily appreciated.

The manner these differences could be incorporated in seismic design codes is still under discussion; in the meanwhile, only a broad division in Hill, Lake and Transition zones is considered, as shown in **Figure 9**. Nevertheless, the knowledge of the form of site-specific spectra allows designers to make proper decisions about the height and stiffness of buildings, in order to avoid near-resonance conditions.

PERFORMANCE OF SOME STRUCTURAL SYSTEMS

During the 1985 Mexico Earthquake some structural systems performed better than reported for strong earthquakes in other countries, while other systems performed worse than expected. Differences are due to peculiarities of the ground motion as well as of the construction practice in Mexico.

Monumental structures built with stone masonry walls and heavy domed roof suffered little damage in Mexico City, despite their inherent weakness to seismic loading. This can only be partially attributed to the fact that their fundamental period of vibration is low (from 0.1 to 0.5 seconds) and far from the peak of spectral ordinates for the lake zone of the city, where they are mainly located. The main reason of the good performance is believed to be the radiation damping by which most of the energy input of the ground motion is returned to the soil by the vibration of these massive buildings.

Another type of structure that performed remarkably well was the load-bearing masonry wall structure typical of single and multi-family low cost housing buildings up to five stories. These buildings are commonly very regular, both in plan and elevation, and possess a good amount of walls in both directions. Walls are usually built with a system called confined masonry, where light reinforced concrete members called tie-columns and tie-beams, surround the walls providing continuity between transverse walls and between walls and floors. They also provide some flexural capacity to the walls, both for in plane and for out of plane loads, and also give them the capacity to sustain significant lateral and vertical loads for quite large lateral displacements, well in excess of those corresponding to the first diagonal cracking (Meli, 1994). As it can be observed in **Figure 10,** the lateral capacity can be sustained up to drift ratios of about 0.005. Confined masonry structures have performed well not only in Mexico City but also in epicentral areas where the ground motion has greater accelerations and shorter prevailing periods.

Reinforced concrete structures, not fully complying with present strict requirements for ductile behavior, had a very poor performance in 1985. The great lateral flexibility of the structural systems commonly adopted were responsible for large lateral displacements and for long vibration periods which, frequently, placed the structures in the peaks of the spectral response. Irregular structural layouts, lack of symmetry both in plane and elevation and sharp changes of lateral stiffness and strength, greatly increased buildings vulnerability. Lack of proper detailing for ductile behavior made the structures more prone to brittle failures, especially due to column shear and eccentric compression; therefore, their behavior degraded very sharply under the long duration motion imposed.

A very common structure for mid rise buildings was the waffle flat-plate floor system, where columns are connected by shallow beams embedded in the slab depth (usually 300-400 mm). Performance of these structures was particularly poor, mainly due to their excessive lateral flexibility and to the low capacity of columns to resist interstory shear forces. Local failures in slab-column connections were also rather frequent. As can be seen from test results shown in **Figure 11**, lateral flexibility of these system, which is already very small since the initial stages of load, is further reduced by cracking in slab-column connection, that occurres at very early stages in the process of lateral loading. The main shortcoming of this system can be overcome if it is combined with other structural elements able to provide the required lateral stiffness, like shear walls, braces or strong and stiff facade frames. Special detailing of the slab-column connection to avoid brittle failure is also required. With these modifications the system is still being used in Mexico.

STRUCTURAL REHABILITATION

The large amount of structural damage and the drastic changes in building regulations requiring a much greater seismic capacity, have originated a large activity of structural rehabilitation in Mexico City as well as in some other areas of high seismic risk in the country, like Acapulco and Guadalajara.

At least 2000 building structures have been strengthened after 1985. Among them, over 1000 schools and a large number of health care and communication facilities. Most of these works corresponded to buildings that did not suffer significant damage due to the earthquake, but that were upgraded to comply with the level of seismic safety implied by the new codes. The compliance of the new code requirements was made mandatory for critical buildings, like hospitals and schools. Since the lateral load capacity required by the new code for reinforced concrete structures in the lake zone was more than twice that of former codes, the rehabilitation of existing buildings implied drastic modifications of the resisting structure.

Most buildings were rehabilitated by rather conventional techniques, like the addition of shear walls or steel braces. Jacketing of columns and beams was also extensively used, mainly in addition to shear walls and bracings. Special attention was to be given to the connection between new structural members and the original structure, and to the displacement capability between old and new structures.

Some less known techniques that showed to be particularly effective are the following.

a) Strengthening of masonry walls with a welded steel wire fabric nailed to one or both wall faces and covered with a cement mortar. In low rise buildings, lateral stiffness was mainly provided by brick infill walls. The wire mesh reinforcement was a simple and effective way to increase strength and stiffness to the levels required by the new code.

b) An innovative bracing technique used postensioned steel cables connected to exterior beam-column joints. This scheme was extensively used to rehabilitate school buildings up to four stories high (Riobóo, 1996). Its main advantages are the negligible increase in building weight, minimum interference with the building operation and the ease tuning of prestress to adjust system strength and stiffness using the capacity of the original structure. The system has been experimentally evaluated by shaking table test at the University of California, Berkeley (Miranda and Bertero, 1990)

c) Addition of energy disipation devices. This is particularly attractive for mid to high rise buildings in the lake zone of Mexico City. The solution is particularly suited to buildings in near resonance conditions with the prevailing vibration period of the soil at the site. A few buildings have been rehabilitated with this technique, making use of the energy dissipation provided by yielding of steel plates (Martínez R., 1996).

d) Uprighting of buildings by underexcavation. Several tall buildings showed severe out-of-plumbness after the earthquake, mainly because of non symmetric settlements produced by overturning moments. Through a controlled extraction of soil under the highest parts of the foundation, a slow rate of rotation of the foundation mat was produced, until a satisfactory correction of the tilting was achieved.

FINAL REMARKS

The experience of the 1985 earthquake has produced significant changes in the design and construction practice, especially in Mexico City, but also in the rest of the country. Building codes have been modified towards stricter requirements for earthquake resistance, and towards more effective ways to control the construction quality. A new building code incorporating the most recent knowledge on seismic risk in Mexico City and advances in earthquake resistant design, is under preparation and is expected to be issued in 1999.

Since 1976 the structural requirements of the Mexico City Building Code are formulated in a limit state format, where loads, load factors and some general limit states are common to all structural materials. The aim of the new version of the seismic code is to arrive at a rational derivation of the main requirements and design parameters, based on performance concepts as those proposed by Bertero (SEAOÇ, 1995).

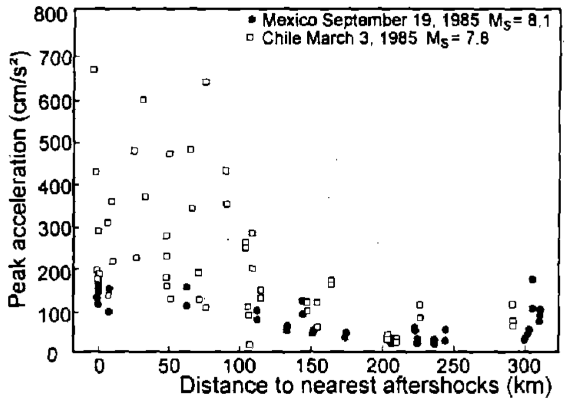

Figure 1. Comparison of maximum ground accelerations generated by the Mexico and Chile Earthquakes (after Rosenblueth and Ovando, 1993)

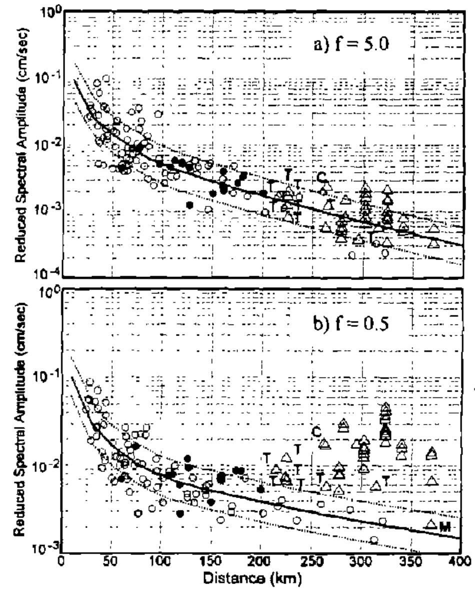

Figure 2. Attenuation of seismic wave amplitudes as a function of hypocentral distance, R, for high and low frequencies. Data from 8 earthquakes. All stations located in firm soil. (After Ordaz and Singh, 1992)

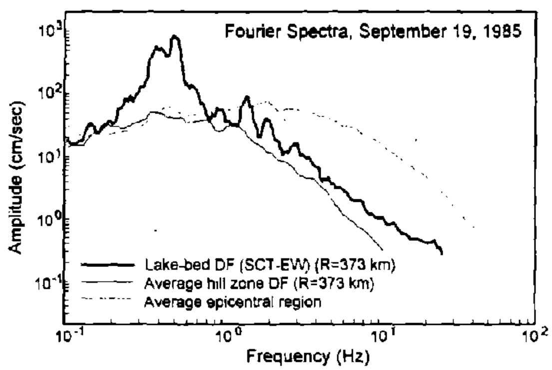

Figure 3. Comparison of average Fourier amplitude spectra in epicentral zone, in Mexico City hill zone and in Mexico City lake zone (Adapted from Ordaz and Singh, 1992)

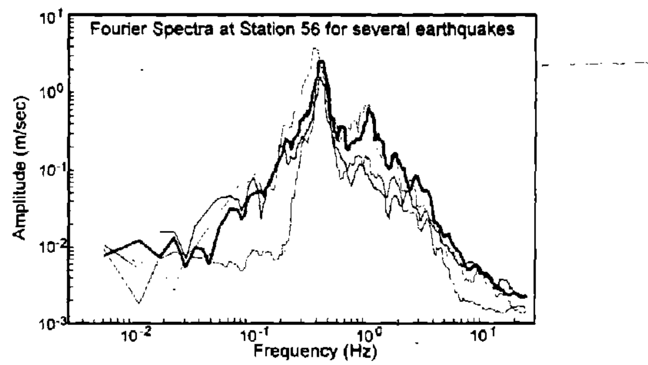

Figure 4. Fourier Spectra from different earthquakes for a particular site in the lake zone of Mexico City (Calculated by Ordaz)

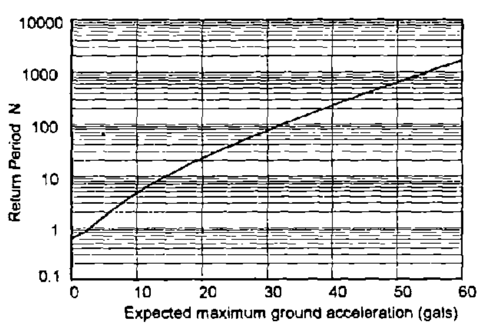

Figure 5. Return Period of maximum acceleration in a site of firm ground in Mexico City (calculated by C. Reyes)

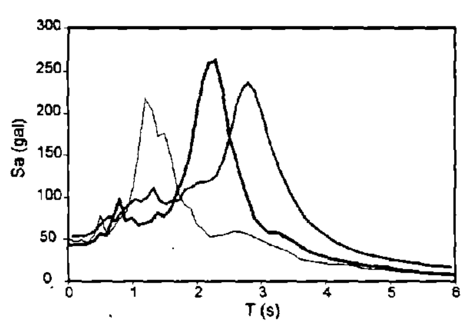

Figure 6. Acceleration Response Spectra for three sites of the lake-zone, from records of the April 25, 1989 Earthquake (calculated by C. Reyes)

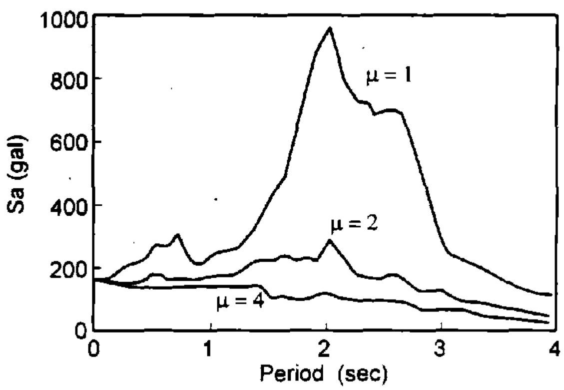

Figure 7. Acceleration Response Spectra for elasto-plastic systems with different ductility coefficients (After Miranda, 1996)

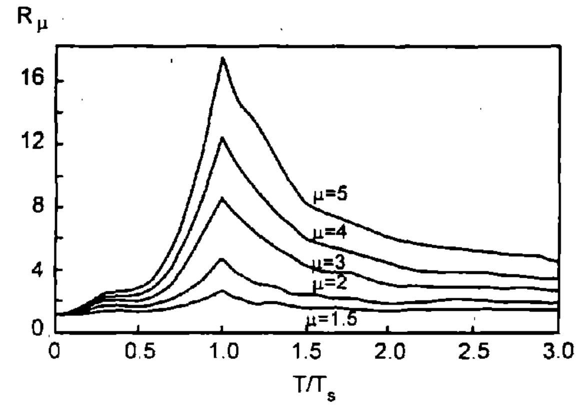

Figure 8. Ductility reduction factors for elasto-plastic systems submitted to ground mottions typical of the Mexico City lake zone (After Miranda, 1996)

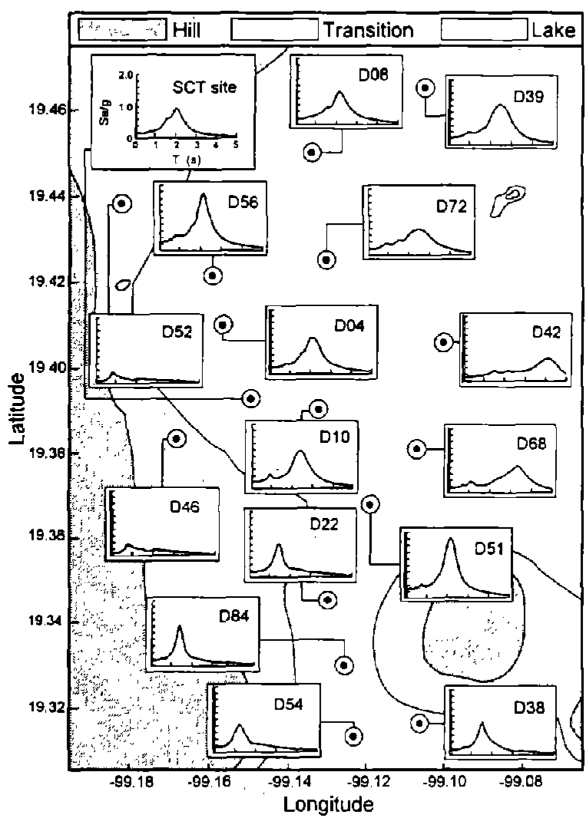

Figure 9. Acceleration Response Spectra calculated for different sites of Mexico City due to a postulated critical earthquake (Ms=8.2, in front of the Guerrero Coast) (Calculated by L.E. Pérez-Rocha)

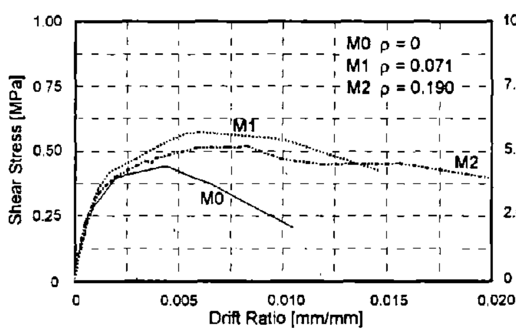

Figure 10. Behavior under lateral loading of confined masonry walls with different ratios of horizontal reinforcement (ρ) (After Aguilar et al, 1996)

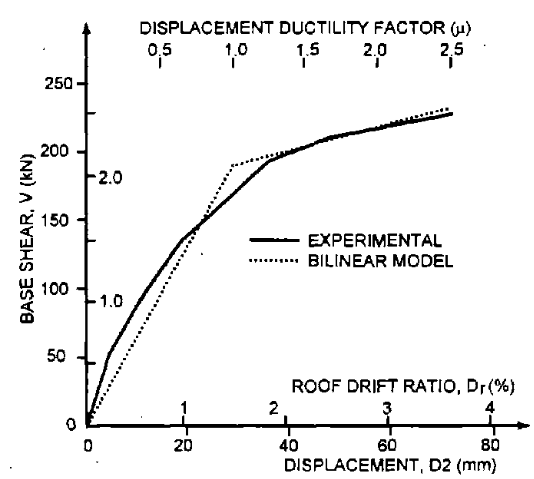

Figure 11. Envelope of experimental lateral load versus lateral displacement for a waffle flat plate test structure (After Rodríguez et al., 1995)

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Important Lessons from Northridge

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SUMMARY

Analysis of the damage patterns in the Northridge earthquake indicate that although code improvements can always be made, improved implementation of the code, both in design and construction, may be the most effective way to improve seismic performance in the short term. The most effective long term improvement will be to develop financial incentives for owners and developers to seek higher levels of seismic performance. Development of consensus performance based engineering methods will accelerate this process.

SUMMARY OF RESULTS OF THE NORTHRIDGE EARTHQUAKE

The Northridge earthquake had a moment magnitude of 6.7 and struck at 4:31 a.m. Pacific Standard Time on January 17, 1994. A summary of vital statistics for this event, and a comparison with the 1989 Loma Prieta earthquake is shown in Table 1 (Holmes, 1994). Accurate dollar loss figures will never be known, but some recent estimates place the figure well over $20 billion, even as high as $30 billion. The casualty figure shown in **Table 1** is also deceptive due to the early morning time of the event, when most people were in the homes or residences. An estimate of 600 to 800 casualties for a 11:00 a.m. occurrence was made in 1994, considering the specific incidences of hazardous conditions (Holmes, 1994).

TABLE 1
SUMMARY OF STATISTICS FOR THE NORTHRIDGE AND LOMA PRIETA EARTHQUAKES

Item	Northridge	Loma Prieta
Casualties (killed)	61	62
Property Damage	$17 billion	$6 billion
Damaged/Impacted Residential	85,000 (City of LA)	22,000
Damage/Impacted Commercial	6,200 (City of LA)	1567
Homes Destroyed	2000 (City of LA)	1000
Psons Displaced	50,000 (City of LA)	12,000
Applications for Disaster Assistance	500,000	80,000

Noteworthy damage to buildings from this earthquake is summarized below:

1. Twenty concrete parking structures were significantly damaged, many of them of recent design and construction.

2. Thirty to fifty older nonductile concrete structures were severely damaged, including several partial collapses.

3. Several hundred partially or completely retrofit URM buildings suffered damage that caused dangerous debris to fall to the street or threatened the stability of the buildings themselves.

4. About 200 steel moment frame buildings, built since the mid-seventies, and including buildings under construction at the time of the earthquake, suffered damage at the beam column joints, ranging from minor cracking in the flange welds to complete fracture of the beam or column in the joint area.

5. The diaphragm-to-wall connection in hundreds of tilt-up buildings deformed excessively or fractured, allowing the wall panels to fall over, and in many cases causing the roof to locally collapse.

6. Damage to inexpensively built wood apartments was extensive. In many cases, the damage, including collapse, was caused by the incorporation of "tuck under" parking at the ground level, creating a soft or weak story. Damage was also noted in similar wood apartments built over one level concrete parking garages.

7. Several thousand single family residences were destroyed, or damaged beyond repair. Except in the cases of hillside homes, this damage seldom created a risk to life safety. Tens of thousands more residences suffered considerable damage, causing some to wonder if single family residences shouldn't be better designed to allow continued occupancy following a damaging earthquake.

ELEMENTS OF THE DESIGN AND CONSTRUCTION PROCESS THAT COULD AFFECT SEISMIC PERFORMANCE

Although the majority of buildings performed well in the earthquake, many policy makers and engineers felt that the performance of one or more of the building types listed above was inadequate. As suggested by performance based engineering documents such as Vision 2000, *all* aspects of the design and construction process should be examined to determine the most efficient short- and long-term improvements.

One possible breakdown of the design and construction process is shown below. Each element should be considered for potential revisions to practice that could result in significant improvement in seismic performance.

1. Conceptual Design
    a) Configuration
    b) Materials
    c) Systems
    d) Performance Intent
2. Design
    a) Criteria (Code)
        i) Demand (ground motion)
        ii) Supply (element strength and deformation capacity)
    b) Understanding of concepts beyond prescriptive provisions
    c) Quality of Implementation
3. Review
    a) Conceptual (peer review)
    b) Detailed (plan check)
4. Construction
    a) Contractor
        i) Understanding
        ii) Skills
    b) Engineer of Record
        i) Drawing Interpretation and follow through
        ii) Field Observation
    c) Inspectors
        i) Jurisdictional
        ii) Other (Special Inspectors, etc.)

The influence of these elements on damage is rated in **Table 2**. The contents of each cell of this table is debatable and requires explanation that is beyond the scope of this paper. However, the table is a useful overall tool to understand the interaction of the various elements and to consider various options to achieve cost effective improvements to the design and construction process.

Although improved practice in all of these areas would result in better seismic performance of buildings, the results of this analysis would point to problems with *implementation of the code* as possibly the largest contributor to building damage in this earthquake. Interrelated aspects include conceptual design and jurisdiction review (plan check), both of which are important in obtaining a building that conforms with the intent and letter of the code. The problems with steel moment frames may be the only pure code related issue; failures in tilt-up wall connections could also be blamed on inadequate code requirements and hardware approval methods, but it could also be argued that better implementation of the current requirements would have reduced this damage significantly.

METHODS TO OBTAIN IMPROVED IMPLEMENTATION OF THE CODE

There are many ways to improve correct implementation of the code including continuing education programs, more rigid professional licensing standards and enforcement, more consistent and thorough plan review, and the development of financial incentives for improved seismic performance.

TABLE 2
LEVEL OF INFLUENCE OF VARIOUS ELEMENTS ON DAMAGE TO DIFFERENT BUILDING TYPES

	Building Type						
Element	Parking	Nonductile	Retro URM	SMRF	Tilt-up	Wood	Residences
Conceptual Design	high	? Note b	low	moderate	low	high	low
Design Criteria (Code) Note a	low	?	low	high	high	low	low
Implementation of code	high	?	high	low	high	high	moderate
Conceptual Review	moderate	?	low Note c	moderate	low Note c	low Note c	low Note c
Jurisdictional Review	low	?	high	low	moderate	high	moderate
Construction Contractor	moderate	?	moderate	low	high	low	high
Construction EOR	moderate	?	high	low	high	low	moderate
Construction Jurisdictional	low	?	high	low	low	low	high
Construction Inspectors	low	?	low	moderate	low	moderate	high

Note a. This analysis is for the purpose of judging the current situation. Problems in older buildings due to older codes are discounted.

Note b. The primary issue for these buildings is use of a inadequate code. Other contributing factors have not been studied.

Note c. Conceptual or peer reviews are so unlikely or impractical for these building types as to not be considered.

Much of the damage that can be attributed to poor code implementation occurred in the most competitive design and construction environments: parking structures, retrofit of URMs, tilt-ups, wood apartments, and residences. "Competitive" engineering in these areas reduces (eliminates?) conceptual engineering time, reduces thoroughness in calculations and details, places high value on low initial construction cost, and encourages repetition of low cost configurations and details that may contribute to poor performance. It is questionable whether, on their own, continuing technical education requirements or rigid enforcement of licensing standards are realistic balances to the strong competitive forces in the construction industry.

However, since it is well accepted that every building needs a building permit (at least in most high seismic zones), more consistent and thorough plan review would improve the correct implementation of the code despite competitive pressures. Such consistent review would soon raise the minimum expected level of engineering effort because it would be accepted that increased effort is required to obtain a building permit. It is well accepted that rigid plan checking has been a major contributor to the high level of performance of schools in California. A similar, but probably less vigorous, program, if instigated statewide and consistently, would have a similar effect on construction in general. Since local communities find it difficult to set a high budget priority on plan checking and construction inspection, a state law will probably be necessary to achieve consistent, thorough, and professional plan checking..

In the long term, the creation of financial incentives to achieve improved seismic performance may naturally force owners and developers to consider the advantages of more thorough initial engineering. If both new and existing buildings carried an expected performance rating, developed in conjunction with performance based engineering concepts, insurance rates, financing terms, rental rates, and eventually the value of the buildings themselves will be influenced. Although minimum acceptable performance--presumably life safety--will be required by the code (and implemented by plan check), higher standards will be valued and often chosen, based on financial analysis. Similarly, existing buildings with expected poor performance will be reduced in value, creating natural incentive for retrofit or replacement. It is currently unclear what standards will be used for building ratings or what agency will approve and maintain ratings, but the creation of such a system should be encouraged by the earthquake engineering community

CONCLUSIONS

In the short term, the most effective and efficient means of improving design of buildings for seismic performance is to institute a program of consistent and throrough plan check in all jurisdictions. In the long term, the development of a consensus seismic performance rating system that would influence building values would create a natural and sustained demand for better performance and improve understanding of the seismic issues in the non-technical community.

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The Impact of Nonstructural Damage on Building Performance: Reflections on the 1994 Northridge Earthquake

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SUMMARY

Nonstructural damage caused by the Northridge earthquake contributed to unprecedented financial losses and rendered structurally sound buildings unusable. Despite the impact of the damage, the primary attention of public and professional groups after the earthquake was on structural damage and effecting changes to structural design practice. Progress related to nonstructural hazard mitigation has been markedly slower. In fact, no substantive changes to design practice have been implemented to reduce future financial losses and prevent the loss of essential services caused by nonstructural damage.

The Northridge earthquake demonstrated that effective nonstructural hazard mitigation requires the collaborative efforts of all design professionals and changes to design, manufacturing and construction practices. The increased responsibility of design professionals, explicit consideration of equipment fragility, preparation of project-specific designs, and systematic post-earthquake studies of nonstructural damage are recommended. The reliable application of performance-based engineering principles and the desire to achieve enhanced performance objectives will require increased attention to nonstructural components.

BACKGROUND

As testimony to lessons learned from past earthquakes, the number of earthquake-related deaths and injuries has dramatically reduced in the United States during the past century. Despite the improved safety offered by modern construction, however, the economic impact of earthquakes is rising.

The unprecedented $30 billion loss attributed to the Northridge earthquake was a result of damage, both structural and nonstructural, and associated loss of operations.

Structural damage received the greatest attention after the earthquake. Front page coverage of collapsed garages and apartment buildings captured the nation's attention. The engineering community was quick to study the implications of structural damage and modify related building code requirements.

The level of public support and professional commitment to learning the lessons related to nonstructural damage was markedly less. Because the damage associated with nonstructural performance was not responsible for significant life loss, it did not capture the same attention from the media. And because the damage to nonstructural components crossed traditional professional bounds, no single professional group felt responsible for addressing the related damage. Yet as the earthquake demonstrated, the damage associated with the performance of nonstructural items such as piping, equipment, cladding and contents can cause huge financial losses as well as the loss of essential post-earthquake services.

This paper is written to focus on the damage to selected nonstructural components in the Northridge earthquake, draw attention to the unique aspects of nonstructural hazard mitigation and offer recommendations for future improvements.

PERFORMANCE OF NONSTRUCTURAL COMPONENTS IN BUILDINGS

WATER PIPING SYSTEMS

The single most disruptive type of nonstructural damage in the Northridge earthquake was breakage of water lines inside buildings, including fire sprinkler, domestic water, and chilled-water systems. Such damage accounted for countless building closures and financial losses. According to a survey of office buildings in the effected area, 1 in 5 buildings suffered water damage (Durkin, 1994).

Perhaps the most dramatic example of the impact that water damage can pose is demonstrated by the experience at the Los Angeles County Olive View Hospital. The hospital, located in Sylmar, California, replaced the Olive View Hospital that was damaged and demolished following the 1971 San Fernando earthquake. The hospital was designed in 1976 and constructed in the mid-1980s.

The epicenter of the Northridge earthquake was located approximately 16 km from the hospital. A peak ground acceleration of 0.8g was measured at the site and 1.5g was recorded at the roof of the six-story steel framed structure.

Despite the fact that the hospital was designed and constructed in accordance with modern standards for essential facilities, the hospital was evacuated after the Northridge earthquake. The sole source of damage that lead to evacuation was water. Specifically, 2 chilled water lines broke at the location of cast iron flanges in control valves and 80% of the piping connected to reheat coils broke and leaked (Ayres & Ezer, 1996).

Similar types of damage were reported throughout the region. Damage was generally attributed to excessive pipe movements and differential deflections between the piping and connected equipment (with failures almost always occurring at fittings). In some instances water damage came from pipes crossing seismic separations between buildings without proper protection to allow for movement. Damage to fire sprinkler systems was often caused by local damage at the sprinkler heads. Relative movement between the sprinkler heads and adjacent ceilings often caused damage to sprinkler piping connections or to the head allowing water to be released. Other sources of damage to sprinkler lines was reportedly caused by the improper use of C-clamps and powder-driven fasteners, and inadequately installed lag screws (Fire Sprinkler Advisory Board, 1994).

EQUIPMENT

Despite long standing code requirements for anchorage of equipment and a general recognition that unless equipment is properly restrained against movement it can and will move during an earthquake, unanchored and inadequately anchored equipment was uncovered by the Northridge earthquake. Damage to equipment was not limited to older facilities - both old and new buildings were among those disabled by inoperable equipment.

As expected, spring-isolated roof mounted equipment was especially vulnerable. Also vulnerable was equipment that was designed with proper restraints but that was installed improperly. One such instance was encountered at the Olive View Hospital where motor control centers were installed with angles connecting to walls at the top but without anchorage at the base (which was required by the design documents). During the earthquake, the units kicked out at the base, overloading the top connections that ultimately pulled out of the wall. In another case at Olive View, chillers were bolted without adequately engaging the supporting vibration isolators and "jumped off" of the isolators during the earthquake.

Damage to pipes connected to anchored equipment was common. Differential movement between the piping and equipment caused the breaks. Flexible connections between isolated units and rigid piping or conduit were generally successful. A more widespread use of flexible connections would have prevented much of this kind of damage.

IMPROVING NONSTRUCTURAL HAZARD MITIGATION

In a hospital water damage study commissioned by OSHPD following the Northridge earthquake it was reported that "...many of the failures identified in the study were similar to those reported after the 1971 San Fernando earthquake," (Ayres & Ezer, 1996). Clearly the opportunity for improving the performance of nonstructural components in earthquakes is great.

The Northridge earthquake demonstrated that efforts to engineer anchorage for nonstructural elements are generally successful when coordinated with all involved design professionals and when properly implemented during construction of the facility. However, when efforts are uncoordinated or construction quality is lacking, all such efforts are rendered futile.

One of the reasons for the lack of coordination and follow-through is directly related to the fact that no single design professional serves as watchdog over nonstructural hazard mitigation. That is, while each discipline has responsibility for design of various building systems and components, depending on contracted responsibilities and regulatory requirements, that same discipline may or may not have responsibility for designing and overseeing the incorporation of seismic restraints. Design professionals are often all too eager to hand off the seismic restraint of nonstructural components to another discipline or to the contractor. When such responsibility is delegated, so to is the verification that proper installations have been made.

The support and bracing of piping systems on building projects is often handled by specifying industry standards such as SMACNA or NUSIG or other proprietary systems. Special conditions that fall outside the scope of these documents are often overlooked and left undesigned. Application of the standards and interpretation of them is commonly left up to the installer and, when present, an inspector.

General contractors commonly delegate responsibility for pipe installation and bracing to subcontractors responsible for individual piping systems. On any given project it is not uncommon to find several independent and different bracing systems used. Opportunities to coordinate the bracing of multiple piping runs are rarely taken. The subcontractor's installer becomes the most important element in the quality control plan being responsible for interpreting and applying industry standards.

Pipe bracing generally receives little focused inspection by individuals knowledgeable in the seismic performance of buildings and piping systems. When building inspectors are given the responsibility for inspecting pipe installations, inspection often focuses on verifying that braces have been installed within the spacing requirements of industry standards. Misapplications of industry standards commonly pass inspection. Without project-specific drawings defining the nature and scope of pipe bracing required, subtle yet critical deviations from requirements are overlooked.

In order to improve the performance of piping systems in earthquakes the related state of practice will require change. Change will need to start with the assignment of responsibility for pipe bracing. A single design professional will need to oversee coordination among design professionals and verify that implementation is in accordance with intended performance objectives and minimum code requirements.

The problem with misapplication of industry standards also needs to be addressed. One approach being implemented by Kaiser Foundation Health Plan in its Northern California region is to require the general contractor to assume overall responsibility for the bracing of all piping systems. Specifically, Kaiser now requires that the general contractor retain a structural engineer to prepare shop drawings for the installation of pipes and equipment. The design professionals of record for the project are required to review and approve the drawings. The availability of such drawings provides an opportunity for inspectors to reliably verify that installed systems comply with the requirements of the contract. In addition, the design professionals of record are required to walk-through the project during installation of pipe bracing and upon completion to verify proper installations and to provide a final check on any unforeseen conditions that may not have been addressed by the shop drawing process.

With the increased responsibility of design professionals and the preparation of installation drawings, the reliability of nonstructural systems is expected to improve dramatically.

MEETING THE CHALLENGE OF PERFORMANCE-BASED ENGINEERING

In order to achieve enhanced performance objectives the reliability of nonstructural systems will need to be improved. As the Northridge earthquake demonstrated, the current standard of care for designing and constructing essential facilities is not sufficient to prevent closure of facilities, even when the buildings are structurally undamaged.

Proper design and installation of seismic restraints for nonstructural systems as previously discussed are a necessary but not sufficient condition of improved nonstructural performance. In addition to ensuring that systems are protected from excessive movement, the continued functioning of equipment must also be addressed. This will require a change from the current standard of practice in building design which is to consider each piece of equipment as a "black box" and to restrain it against shifting or overturning. In order to protect its ability to function after an earthquake, the black box must be opened and examined.

Unfortunately, at present it is practically impossible to specify equipment that has been reliably tested and proven sufficiently robust to remain functional after an earthquake. With the exception of equipment used in the nuclear industry, equipment manufacturers have not typically undertaken such qualification testing. Thus it is generally economically unfeasible to reliably purchase and install equipment that has a high reliability of performing after an earthquake. Without the introduction of such equipment qualification testing, the reliability of designing to achieve performance objectives that seek continuity of operations is low.

The Northridge earthquake also demonstrated that the ability to resume operations in a building after an earthquake can be extremely sensitive to nonstructural damage. As the EERI Northridge Earthquake Reconnaissance team reported (EERI, 1995), "even in cases where seismic detailing prevented a large amount of damage, a few seemingly small failures (for example, one or two pipe breaks) were sometimes enough to cause large disruptions in some buildings". In order to minimize the probability of nonstructural damage that can interrupt operations, changes in design and construction practice must be accompanied by an improved understanding of the performance of nonstructural systems, particularly piping systems. Timely, systematic post-earthquake studies of piping failures are needed to add to our knowledge base the information needed to improve the performance of these systems.

The Vision 2000 report prepared by the Structural Engineers Association of California (SEAOC, 1995) sets forth interim recommendations for protecting nonstructural building components to attain fully operational or operational performance levels. Additional research is needed to direct industry-wide changes for reliable nonstructural hazard mitigation.

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Needs To Evaluate Real Seismic Performance Of Buildings

-Lessons from the 1995 Hyogoken-Nambu Earthquake-

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SUMMARY

The lessons for buildings from the damage due to the Hyogoken-Nambu Earthquake are necessity to develop more rational seismic design codes based upon a performance-based design concept, and to evaluate seismic performance of existing buildings. In this paper, the lessons for buildings from the Hyogoken-Nambu Earthquake, the building damage due to the earthquake, the reasons why the seismic retrofit has not been implemented much, the responses to the lessons from the earthquake, the Network Committee for promotion of seismic retrofit of buildings, the Law for promotion of seismic retrofit of buildings and the implementation of seismic retrofit in Japan are described.

INTRODUCTION

The lessons from the Great Hanshin-Awaji Earthquake Disaster caused by the 1995 Hyogoken-Nambu Earthquake on building structures could be summarized as follows;
1) Most new buildings designed and constructed according to the present seismic codes showed fairly good performance for preventing severe structural damage and/or collapse, even to such severe earthquake ground motions. However, the problem was that the seismic performance of buildings was widely ranged from the level of collapse preventing to function keeping, which could not have been identified by the present seismic codes. Therefore, it is strongly needed to develop more rational seismic design codes based upon the performance-based-seismic design concept, where the performance of buildings including structural and functional safety during and after earthquake is explicitly explained.
2) Most buildings which took serious damage were those designed and constructed before the present seismic codes adopted in 1981. About 3,000 buildings, of which stories were more than two, collapsed or severely damaged and almost same number of buildings took medium structural damage. The collapsed or seriously damaged ratio was 6.4 % in average and about 15% in the most affected area. Besides them, about 46,000 wooden houses, which was 9.4% in average, were collapsed or severely damaged (**REFERENCE 1**). Therefore, urgent needs of seismic evaluation of their seismic performance to identify seismically vulnerable buildings, which have not experienced severe earthquake ground motion yet, and of seismic retrofit to upgrade their seismic performance have been strongly recognized.

A lot of projects to develop performance-based-seismic design and to carry out seismic upgrading of existing vulnerable buildings have launched since the Hyogoken-Nambu Earthquake. In this paper, the emphasis is laid upon the seismic retrofit of existing reinforced concrete buildings which has been considered as one of the most urgent earthquake preparedness since the Hyogoken-Nambu Earthquake.

BUILDING DAMAGE DUE TO HYOGO-KEN NAMBU EARTHQUAKE

As mentioned above, the damage to buildings and houses was serious for those constructed before 1981, especially before 1971, because Japanese seismic design codes in 1950, which was basically same as the first Japanese seismic design codes for buildings in 1924, was revised in 1971 and 1981 (see **Table 1**). In 1971, specifications such as detailing of re-bar arrangement of reinforced concrete members were revised to increase ductility and the consideration of ductility was required in estimating ultimate lateral strength in 1981. In order to promote seismic retrofit of pre-code revision buildings, the standards for evaluation of seismic capacity and guidelines for retrofit of existing reinforced concrete buildings were developed and published from the Japan Building Disaster Prevention Association in 1977(see **Reference 2** and **APPENDIX**). However, they have been applied only to a limited number and limited types of buildings in a limited areas excluding the Hanshin-Awaji area.

Table.2 is a statistic showing the relationship between the damage grades due to the Hyogoken-Nambu Earthquake and the construction years of reinforced concrete buildings in a part of Kobe city. The ratio of severely damaged buildings constructed before 1971 is much higher than new buildings (**REFERENCE 1**). **Fig.1** is another example showing the similar tendency for reinforced concrete school buildings, where the vertical axis is showing the damage grade index of each building and the horizontal the construction year. No school buildings constructed after 1982 suffered from serious damage (**REFERENCE 3**).

WHY THE SEISMIC RETROFIT HAS NOT BEEN IMPLEMENTED MUCH ?

Even though the necessity of seismic retrofit had been pointed out before the Hanshin-Awaji Earthquake Disaster , why the seismic retrofit has not been implemented much ? The reasons could be summarized as follows;

1) The seismic retrofit is less attractive for owners, architects, engineers, researchers, constructors, administrators and politicians than new building construction.

2) Since a return period of a big earthquake is usually long, owners are apt to hesitate to spend money in seismic retrofit of existing buildings.

3) Since a seismic retrofit is more complicated than construction of new buildings, it is usually troublesome for architects and engineers, and less paid.

4) Since the Japanese Building Code is not retroactive, a seismic retrofit is not enforced by law.

RESPONSES TO THE LESSONS FROM THE GREAT HANSHIN-AWAJI DISASTER

Since the Great Hanshin-Awaji Earthquake Disaster was a great shock to Japanese people, various responses have been quickly taken to upgrade the seismic capacities of pre-code revision buildings all over Japan. In order to promote the seismic retrofit, it is necessary 1) to develop methodologies to evaluate seismic capacities, 2) to develop techniques to strengthen existing buildings, 3) to train engineers, and 4) to prepare subsidies, low-interest loan, tax exemption and so on, to increase public incentive for retrofit.

Some of the major responses are; 1) The notices to recommend seismic capacity evaluation and seismic retrofit of pre-code revision buildings and houses were issued by the Ministry of Construction in March 1995, 2) Network Committee for promotion of seismic retrofit of buildings was established in April 1995, 3) Architectural Institute of Japan published the recommendations reflecting the damage due to Hyogoken-Nambu Earthquake including the importance of seismic retrofit in July 1995, 4) Japan Basic Plan for Disaster Mitigation was revised emphasizing the importance of seismic retrofit by Land Agency in July 1995, and 5) Law for Promotion of Seismic Retrofit of Buildings was enforced in December 1995.

In the followings, the activities of the Network Committee, the Law for Promotion of Seismic Retrofit of buildings, and the Method for evaluation of seismic performance of existing reinforced concrete buildings are briefly described;

NETWORK COMMITTEE FOR PROMOTION OF SEISMIC RETROFIT OF BUILDINGS

The network committee chaired by the author consisting of 76 organizations related to the design and construction of buildings and houses including associations for academic people, for architects, for engineers, for consultant offices and for building owners. Major activities of the Committee are; 1) to exchange information on seismic retrofit, 2) to organize seminars to train engineers, 3) to support local governments and groups of engineers to establish local centers for promoting seismic retrofit, and so on.

More than 12,000 engineers attended the seminars in last two years, while they were only about 2,500 for 15 years before the Hyogoken-Nambu Earthquake, and more than 60 local centers have been established. One of the major activities of such local centers is to organize committees to review the results of evaluation and retrofit design by engineers, which may also contribute to improve the level of engineers.

LAW FOR PROMOTION OF SEISMIC RETROFIT OF BUILDINGS

The objective of the Law is to enforce the seismic retrofit on the owners of the specified occupancy and/or large occupants buildings and to prepare the incentives to implement seismic retrofit of other buildings and houses. For this purpose, the law identifies the important buildings such as schools, hospitals, department stores, theaters, office buildings and so on, which occupy a large number of inhabitants and visitors, and enforces the owners to make seismic retrofit. If the building officials approve the retrofit plans of such buildings, the owners are eligible to apply lower interest loan, tax exemption, and exemption from regulations for land use and fire protection codes. The owners of other types of buildings and houses may have similar eligibility.

IMPLEMENTATION OF SEISMIC RETROFIT

It is assumed that there are about 18 million wooden houses and more than 2 million buildings which were designed and constructed by the previous seismic codes. Considering the damage due to past earthquakes including the Hyogoken-Nambu Earthquake, about 20 percent of wooden houses and 10 percent of buildings are assumed to be vulnerable. Therefore, a lot of retrofit works have been going on since the Great Hanshin-Awaji Earthquake Disaster.

In order to evaluate the seismic capacity and retrofit of existing reinforced concrete buildings, the Evaluation Standard and Retrofit Guideline for Existing Reinforced Concrete Buildings (**REFERENCE 1**) have been widely used since 1977.

The procedure to judge the seismic performance of existing building by the Evaluation Standards is as follows;
First, the seismic index of Is is estimated to evaluate the seismic capacity of the building, then it is compared with the judging index of Iso. If the Is index is larger than the Iso index, the building is judged to have good seismic performance.
This standard was applied to the reinforced concrete buildings which suffered from the 1968 Tokachi-oki Earthquake, 1978 Izuoshima-Kinkai Earthquake, and 1978 Miyagiken-oki Earthquake, and it was clarified that most buildings of which Is indices were less than 0.3 took severe or moderate damage and the damage was slight for building of which Is indices were greater than 0.6. Therefore, Is of 0.6 has been recommended for judging criterion (**REFERENCE 1, 4 and 5**). Similar study was carried out for reinforced concrete school buildings suffered from the 1995 Hyogoken-Nambu Earthquake and it was found that the Is index of 0.6 is almost border between severe damage and moderate damage (**REFERENCE 3**). Considering these studies, the judging index of 0.6 is adopted in the Law for Promotion of Seismic Retrofit of Buildings as a standard criterion to prevent collapse or severe damage. The law says 1)if the Is index is greater than 0.6, the building may have a low possibility of collapse or severe damage, and if the Is index is less than 0.3, the building may have a very high possibility of collapse or severe damage.

It is a hard task and takes a long time to complete the retrofit of vulnerable buildings and houses, however, it should be implemented to mitigate disaster due to future earthquakes.

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4. Okada, Tsuneo, Umemura, Hajime and Murakami, Masaya, "Judging Index for Seismic Capacity Evaluation of Reinforced Concrete Buildings", Proceedings of Annual Convention of Architectural Institute of Japan, September, 1980
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APPENDIX

The Evaluation Standard basically judges the building safety based upon the following structural index(Is) :

$I_S = E_0 S_D T$

where, E_0 : basic structural index, estimated by Strength Index (C), Ductility index (F), and Story Index () at each story and each direction when the story or building reaches at the ultimate limit state due to lateral force.

C : index of story lateral strength, estimated by ultimate story shear in terms of story shear coefficient.

F : index of story ductility, estimated by ultimate deformation capacity normalized by the story drift of 1/250 when a standard size column is assumed to fail in shear. For most ductile column, F is assumed as 3.2 and for short and extremely brittle column, F becomes smallest of 0.8.

: index of story shear distribution during earthquake, estimated by the inverse of design story shear coefficient distribution normalized by base shear coefficient.

S_D : modification factor, estimated by stiffness discontinuity along stories, eccentric distribution of stiffness in planes, irregularity of framing and so on, ranging from about 0.5 to 1.2.

T : reduction factor, estimated by the grade of deterioration, ranging from about 0.5 to 1.0.

TABLE 1

HISTORY OF JAPANESE SEISMIC DESIGN CODES FOR BUILDINGS

1924	K=0.1	Allowable Stress Design	Steel : 1/2 of Yield Strength Concrete : 1/3 of Compressive Strength
1950	K=0.2	Allowable Stress Design	Steel : Yield Strength Concrete : 2/3 of Compressive Strength
1971	K=0.2	Allowable Stress Design	Steel : Yield Strength Concrete : 2/3 of Compressive Strength
		Ductility Requirement	Ultimate Shear Strength > Bending Strength Specifications: Revision of Tie Requirement
1977	(Standards for Evaluation of Seismic Capacity)		Seismic Capacity = Strength x Ductility
1981	C_0=0.2	Allowable Stress Design	Steel : Yield Strength Concrete : 2/3 of Compressive Strength
	C_0=1.0	Ultimate Strength Design	Design Strength = C_0 / Ductility (C_0=1.0)

TABLE 2
DAMAGE GRADE VS.CONSTRUCTION YEARS OF REINFORCED CONCRETE BUILDINGS IN A PART OF KOBE CITY [REFERENCE 1]

	Pre-1971	1971-1981	Post-1981
Collapse or Severe Damage	22 (24 %)	5 (5 %)	3 (6 %)
Medium Damage	8 (9 %)	4 (4 %)	2 (4 %)
Minor Damage	12 (13 %)	12 (13 %)	6 (13 %)
No Damage	51 (55 %)	73 (77 %)	34 (76 %)
Total	93 (100 %)	94 (100 %)	45 (100 %)

FIG.1 DAMAGE GRADE INDEX VS. CONSTRUCTION YEAR OF REINFORCED CONCRETE SCHOOL BUILDINGS DAMAGED DUE TO HYOGOKEN NAMBU EARTHQUAKE [REFERENCE 3]

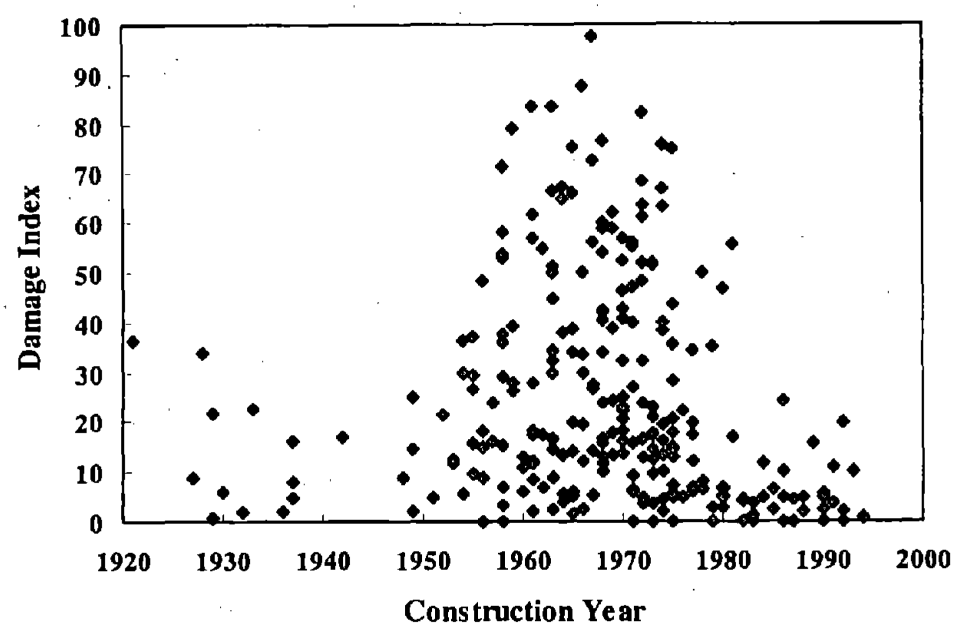
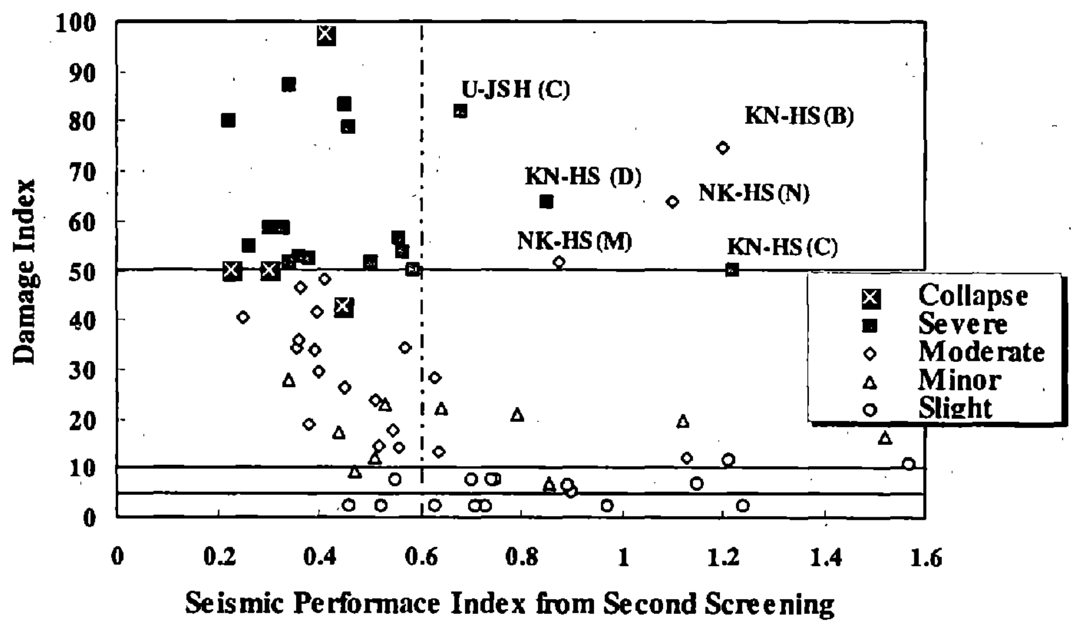


FIG.2 DAMAGE GRADE INDEX VS. SEISMIC INDEX I_S OF REINFORCED CONCRETE SCHOOL BUILDINGS SUFFERED FROM 1995 HYOGOKEN NAMBU EARTHQUAKE [REFERENCE 3]



HANSHIN-AWAJI DEVASTATING EARTHQUAKE

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JAPAN

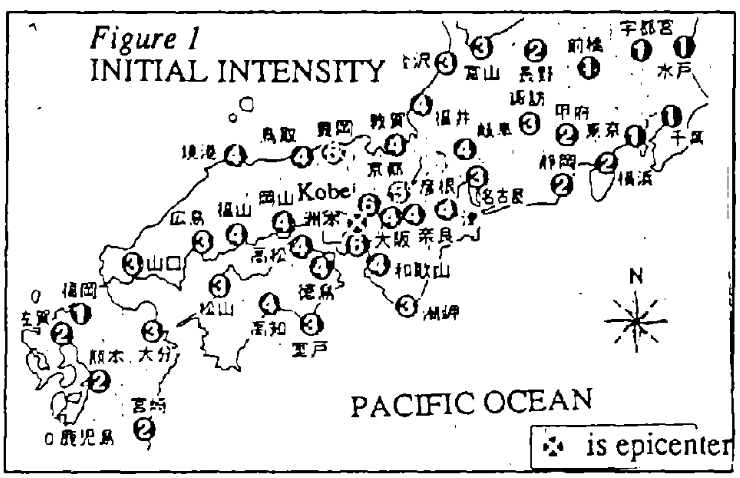
SUMMARY

The January 17, 1995, Hanshin-Awaji Devastating Earthquake occurred in the western part of Japan. The Nojima active fault in Awajishima (near Kobe City) was fractured. It was so called near field earthquake with magnitude 7.2 (less than 7.2 U.S.) and the depth of the hypocenter was about 14 km. 6308 people died and about 30,000 people were injured. Also about 100,000 buildings, including wooden houses, collapsed. It was the largest natural damage in this century in Japan. About \$100,000,000 were directly lost and about \$20,000,00 were lost indirectly. At the beginning, the severity of ground motions due to this earthquake were compared to other ground motions due to the other recent earthquakes and the standard earthquakes such as the EL CENTRO Earthquake in linear and non-linear responses. Then the features of R.C. and S.R.C. building structures were explained. The brittle damage features of steel buildings were explained. Finally the severe damage of wooden houses were explained suggesting what kind of wooden houses subjected to the serious damage.

EARTHQUAKE GROUND MOTIONS
THE MAP OF SEISMIC INTENSITY

In Japan, ground motion intensities are generally assigned a numerical value according to the Japanese Shindo scale by the Japanese Meteorological agency (JMA). For this earthquake, values of the Shindo are shown in **FIGURE 1**. The cities of Kobe reached levels of near Shindo 6 intensity, the cities of KYOTO, TOYOKA, HIKONE were assigned the Shindo 5 intensity. Later, the JMA reassigned levels of 6 to 7 for some portions of the cities of KOBE, ASHIYA, NISHINOMIYA, and TAKARAZUKA and portions of the northern part of AWAJI island, after considering the damage sustained by these areas (**FIGURE 2**).

Figure 1
INITIAL INTENSITY



FAULT PARAMETERS

The solution based on the distribution of compression and extension of the initial motion became clear. It can be inferred that the local mechanism is characterized by a N229E strike dip equal to 77 degrees, and slip of 173.

Prof. Kikuchi of the University of Yokohama City reported that bilateral rupture had occurred with the following characteristics:

Focal mechanism: Strike=223 degrees, Dip=85 degrees, Slip=165

Seismic moment: Mo=2.5x1026 Dyne cm

Fault areas: S=40x10km^2.

Relative displacement: U=2.1 m.

Stress drop: D=100-200 bar.

Duration of main rupture: T-11 Sec.

This earthquake was reported to consist of three sub-events; the parameters for these sub-events are shown in **FIGURE 3.**

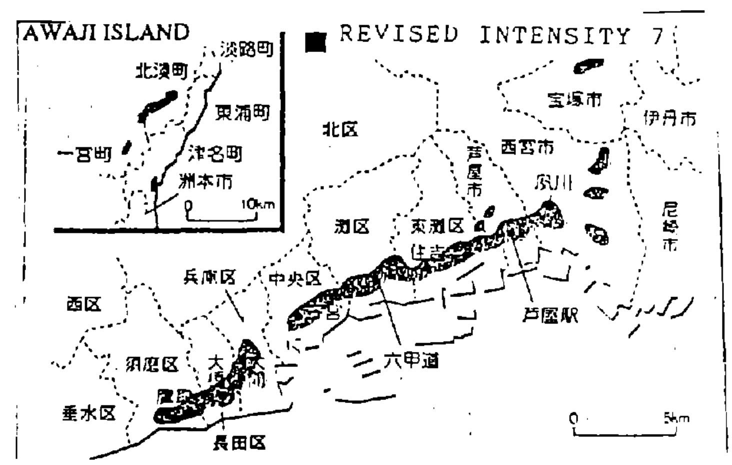
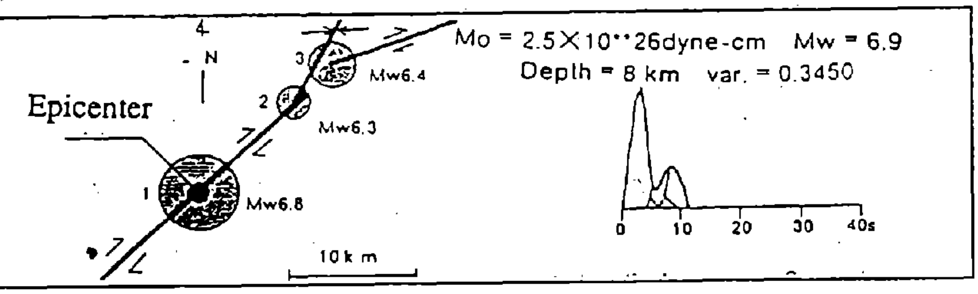


Figure 2 REVISED INTENSITY

Activity of aftershocks: The distribution of aftershocks as of 9:00 am. local time on Jan. 27 had occurred. The surface projection of aftershock activity was along a line approximately 50km long: this length is slightly longer than the 40 km length estimated by Prof. Kikuchi for the main shock. Aftershocks occurred with decreasing frequency after the mainshock. As of 9:00 a.m. local time on Jan. 27, eight aftershocks occurred with maximum intensities of Shindo 4 or greater.

TIME HISTORIES AND RESPONSE SPECTRUM (LINEAR AND NON-LINEAR RESPONSE) OF THE REPRESENTATIVE RECORDED GROUND MOTIONS:

Figure 3 Fracture of Active Faults.



In **FIGURE. 4,** histories of ground motions of three dimensional (North-South, East-West and vertical directions) at Kobe Oceanic Meteorological Agency were indicated. Although the site is not very flat, with some hills about 20 meters high, this was one of the representative earthquake ground motions of the devastating Hanshin-Awaji earthquake. As they indicated, peak acceleration in the North-South directions was 818 gals, 617 gals in the East-West direction and 332 gals in the vertical direction. The duration of the ground motions was quite short, less than

15 seconds, as indicated in time histories. The linear response spectrum of recorded ground motions in the North-South direction at Kobe Oceanic Meteorological Agency are shown in a solid line in **FIGURE 5**. Shown in the same figure, are the response spectra from the TAZANA record due to the Northridge Earthquake in the suburbs of Los Angeles 17 Jan. 1994 and one in the East-West direction from the KUSHIRO Meteorological Agency due to the KUSHIRO Earthquake 15 Jan. 1993. The response spectra indicate that the ground motions from the Hanshin-Awaji Devastating Earthquake are not very severe compared with these response spectra from these other earthquakes. Time histories at the same scale from records at the KUSHIRO Meteorological Agency were shown in **FIGURE 6**. The KUSHIRO Earthquake seems to be a much more severe earthquake than the Hanshin-Awaji Devastating Earthquake. But the KUSHIRO Earthquake is responsible for only two casualties and quite limited damage, despite a peak acceleration of 910 gals greater than that of Hanshin-Awaji Devastating Earthquake. The vertical ground motion spectrum is shown as a solid line in **FIGURE 7**. Vertically, longer periods (from about 0.8 seconds to 2.0 seconds) were recorded in the Hanshin-Awaji Devastating Earthquake with some ground motions exceeding those from the other ground motions earthquakes. In a sense, intensities of vertical ground motions were about half of those in the horizontal ground motions. Although, the intensities of ground motions due to the Hanshin Awaji Devastating Earthquake in linear response were not so severe compared to those due to the Kushiro Northridge earthquakes, the non-linear (Degrading Tri-liner Model suitable for R.C. structures shown in **FIGURE 8 and 9**) response spectra of ground motions, for instance, recorded at the Kobe Oceanic Meteorological Agency due to the Hanshin-Awaji Devastating Earthquake were extremely severe compared to the other ones mentioned above as is shown in **FIGURE 10**. This research was performed by Professor Tadao Minimami of the University of Tokyo.

If the base shear coefficient were to be reduced by one third, as is shown in **FIGURE 11**, rather small peak acceleration records at STC in Mexico City due to the Mexico Earthquake of 1985 become rather severe compared to Hanshin-Awaji Devastating Earthquake ground motions.
The above facts indicate that ground motions due to the Hanshin-Awaji Devastating Earthquake were quite severe motions against rather strong structures to statically lateral force such as shown in **FIGURE 10**. For instance, the base shear coefficient of a 10 storey buildings is 0.5, while those due to the Mexican Earthquake were severe ones against weak structures with statically small lateral base shear coefficient, which is almost one third of the average coefficients in Japan.

SOME FEATURES OF EARTHQUAKE GROUND MOTIONS AS ENGINEERING POINT OF VIEW

Earthquake ground motions due to the Hanshin-Awaji Devastating Earthquake considering building engineering have the following features:
(I) Quite large peak accelerations and peak velocities were recorded in wide districts as near field earthquake. (ii) Periods between 0.8 to 2.0 seconds were especially exceeded. (iii) Large vertical ground motions were recorded. (iv) Duration of ground motions was quite short. (v) Large shaking toward the northern direction was observed. (vi) Non-linear response of vibrations in soft soil areas was observed.

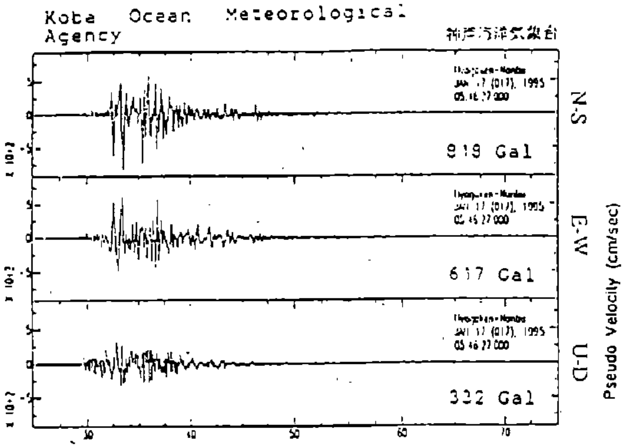


Figure 4. RECORDED TIME HISTORIES DUE TO HANSHIN EARTHQUAKE.

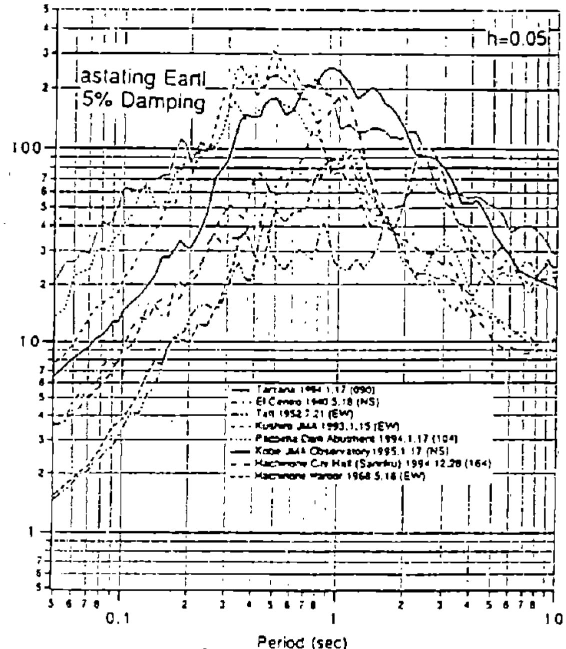


Figure 5.
RESPONSE SPECTRA of KOBE OCEANIC METEOROLOGICAL AGENCY(HORIZONTAL)and OTHERS

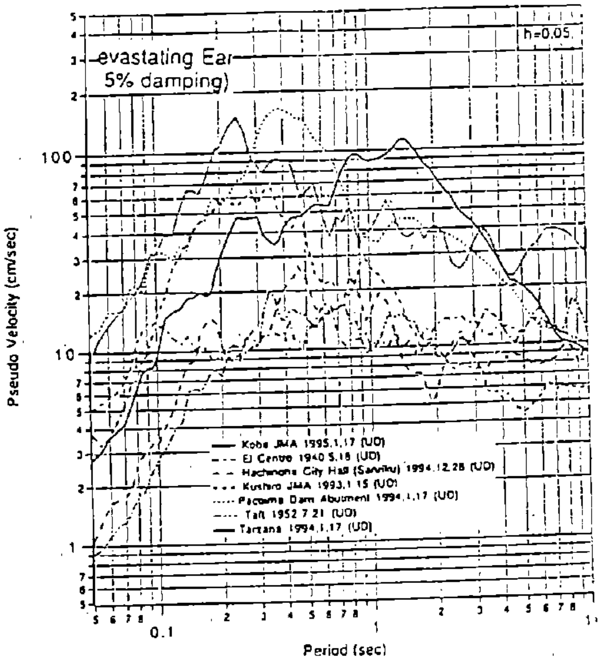


Figure 7.
RESPONSE SPECTRA of KOBE OCEANIC METEOROLOGICAL AGENCY(VERTICAL) and OTHERS.

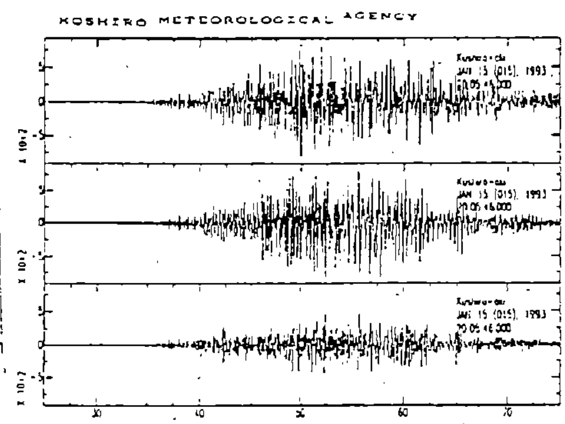


Figure 6 RECORDED TIME HISTORIES DUE TO KUSHIRO-OKI EARTHQUAKE

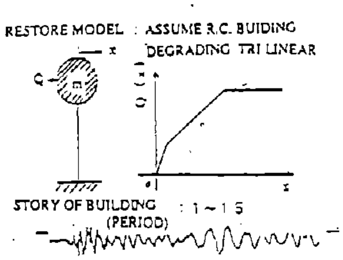


Figure 8
DEGRADING TRI-LINEAR MODEL

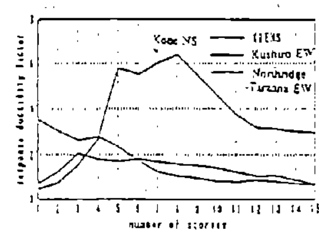


Figure 10
NON-LINEAR RESPONSE
FOR JAPANESE AVERAGE
BASE SHEAR COEFFICIENT

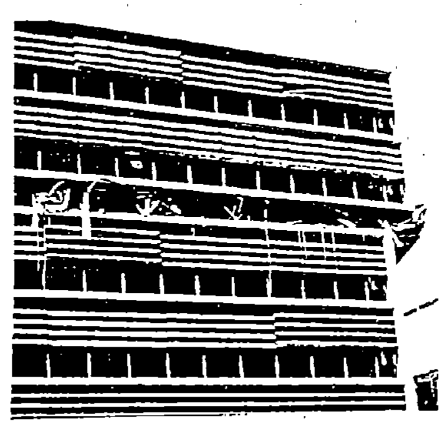


Figure 12
COLLAPSE of INTER-STORIES
of R.C.Structure.

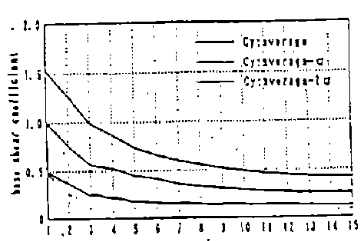


Figure 9
AVERAGE BASE SHEAR
COEFFICIENT of EXISTING
BUILDINGS in SIZUOKA JAPAN

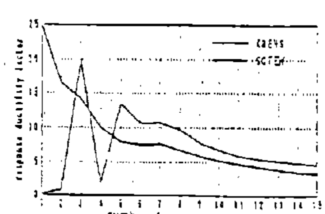


Figure 11
NON-LINEAR RESPONSE
FOR MEXICAN AVERAGE
BASE SHEAR COEFFICIENT

Also, the effect of soil-structures interactions in various districts was observed. Peak accelerations of recorded ground motions near the epicenter (within 40-50km), were ranged from 300 gals to 800 gals, and peak velocities of those were very large (more than 80 cm/sec).

DAMAGE OF REINFORCED CONCRETE STRUCTURES
FEATURES OF THE DAMAGE

The features of the damage of reinforced concrete (RC) and steel-reinforced concrete (SRC) structures are;
(I) The percentage of damage to structures constructed before 1971, when the requirement of hoop interval of column was revised to half, was quite high.
(ii)The damage ratio of the structures constructed after 1971, in reverse, was quite low, especially to buildings constructed after 1981, when the present seismic design codes and regulations for buildings were revised and performed. There was no severe damage to these buildings except those with special types of structures such as soft first stories, which are called PIROTY in Japan. The detailed damage features are:
(I) Collapse or extensive damage to soft first story structures. (ii) Collapse or extensive damage of the first story in buildings with other than soft first storeys. (iii) Complete collapse or extensive damage of an interstorey of the structure. (iv) Damage at the boundary of steel RC composite and RC structures. (v) Damage at connections of steel structure part and anchor connections at the bottom of columns. (vi) Damage at the anchor connections of reinforcing bars of the structural walls of SRC. (vii) Damage at the beam-column connections. (viii) Shear collapse or severe damage of RC columns due to fracture of hoops of the columns (**FIGURE 13**). (IX) Damage due to butting of neighbor structures. (x) Fall-down damage of pre-cast boards of roofs.

Figure13
SEVERE SHEAR FRACTURE of R.C.COLUMN.

Figure 14
QUITE BRITTLE TENSION FRACTURES AT STEEL LARGE SIZE BOX COLUMN.

Wall type RCA structures had almost no damage even though they were constructed before 1981. The main reasons for the above damages are as follows:

(I) Collapse or extensive damage of soft first story structures.

As the name of these type of structure implies, the first story is considerably weaker and softer compared with the other stories. In Japan, these type of structures are called "PIRATE". Essentially, earthquake shaking energy concentrates at the soft deformable top and the bottom of the first story's columns. Then these portions become more flexible due to some damage, so again the energy of the ground motions concentrates more to the same parts of the structure. This phenomenon proceeds in a rather short time and finally destroys the first story of the soft first story, where the most severe shear force is loaded.

(ii) Collapse or extensive damage to the first story of other than soft first story type.

Damage of this kind happened to the building structures constructed before 1981 according to the old seismic design codes. The first story usually loaded the largest shear force due to Hanshin-Awaji Devastating Earthquake ground motions, brittle shear failures occurred in the weakest first story.

(iii) Complete collapse or extensive damage of one of the inter stories of the structures. According to the old seismic design code, the lateral shear coefficient was 0.2 constantly from the ground level up to 16m high. It was different from the actual shaking mode. The present seismic design codes require an Ai distribution mode similar to the real shaking mode.

As such, for an approximately 16m high story, the design lateral shear force and the real one become much different; the former rather less than the actual response force. So the inter story completely collapsed or was severely damaged. Another concept was proposed by Professor Iwan of Caltech. According to him, the deformation of ground motions propagated up to the top of the building and reflected back to the inter story. Meanwhile the next big deformation of the ground motions also propagated up to the same inter story. If the phase were in the same direction, deformation became big enough to destroy the whole inter story.

(iv) Shear collapse or severe damage of RC columns due to fractures of hoops of the columns. In the 1968 Tokachi-oki Earthquake, various RC columns failed due to lateral shear force because the intervals of hoops were, in those days, less than 15 cm; near the beam-to-column connection less than 10 cm intervals were required. Since then, shear strength of RC columns has become at least twice as strong.

DAMAGE OF STEEL STRUCTURE
THE FEATURES OF DAMAGE

The features of damage to steel structures were: both buildings constructed and after 1981 were damaged. Severe damage was mainly to welded portions, high-tension bolts connections parts and base and anchor parts of the columns.

(I) In cases where box type columns were used, damage was found around the base of the columns, as well as fractures of welding at beam-column connections and fractures of column-to-column connections.

(ii) In case of diagonal bracing type structures, fractures of bracings and at the bolt connections occurred, essentially, by tension forces. But in some cases, buckling of bracings was observed by compression forces. And gusset plates for bracing were fractured, as well as fractures of beams due to lack of bending strength.

(iii) The typical damage was due to long duration strains of thin plate steel structures.

(iv) Brittle fractures of columns composed of thick (more than 50 mm) plates with a confined shape - like the box shape or tube shape of rather high-rise residential buildings.

(v) Butting of two neighboring steel buildings.

The main reasons for the above types of damage are as follows:

(i) Damage around the base of the columns. The damage was concentrated in the bare type columns, and fractures of base concrete and of anchor bolts were often observed. Generally, the base of the bare columns were assumed to be pin connections, but the actual base of the columns were not complete pin but rather had limited deformabilities with fixed conditions. So some bending moments were actually worked where strength against bending moment were not considered at the design. 16% of the steel buildings constructed before 1981 were damaged at the bases of columns but only 4% designed after 1981 were damaged.

(ii) Fracture of welding at beam-column connections. These type of damages mainly happened in structures with box type columns and H type beams connected with simple diaphragms, which had almost no plastic deformations. So damage was concentrated to welding portions, such as box column and diaphragm, beam-to-column panel and diaphragm, end of H beam at under parts of flanges and webs as well.

(iii) Brittle fractures of columns composed by thick (more than 50 mm thickness) plates with confined shapes, such as box or tube shapes, of rather high-rise residential buildings. Quite brittle tension fractures occurred at the near welding portions of column-to-column, as were shown in **FIGURE 14**. The reasons for these brittle tension fractures were not so clear, but the most common fractures were occurred at the columns of railway bridges of infra-structures. Some researchers suggest that these kinds of brittle tension fractures occur when the following 3 S conditions occur identically; a) Size (thickness is more than 50 mm, and which of column is more than 50 cm), b) Shape (closed shape such as box shape or tube shape as opposed to an H shape) and c) Speed (speed of strain of the material must be quite high).

Figure 15
COMPLETE COLLAPSE of THE FIRST STORY of WOODEN HOUSE CONSTRUCTED TWO MONTHS BEFORE THE EARTHQUAKE

DAMAGE TO WOODEN HOUSES

NON DAMAGED OR SLIGHTLY DAMAGED WOODEN HOUSES

The following types of wooden houses had little damage even, in areas of intensity 7 of JMA.: (i) Houses which met with the present seismic design codes in which proper construction checks were performed. For instance, houses built using pubic residential funds, which were usually checked for quality of construction. Of the 529 houses in Kobe of this sort, only 1.7 % collapsed. (ii) The traditional type of the wooden house with properly located shear walls. (iii) Houses with two by four frame board walls. (iv) Pre-fabricated wooden houses. (v) Wooden houses of 3 stories for which structural calculations were made.
The main reasons for damage in these types of structures follow:
(I) Most of the collapsed or failed wooden houses had the following similar conditions: (a) Aging construction, with columns and foundations which were rotten or damaged by termites. (b) Very heavy roof tiles covering clay roofs designed for protection against typhoons. And (c) Construction which did not meet the present seismic design codes. (ii) Improper number and location of walls. Essentially, walls were not be located in south direction, because generally this direction was left open, without walls. This natural request made the wooden houses rather weak against earthquake ground motions. **FIGURE 15** shows a typical collapsed first story due to lack of walls and improper location of them only after two months after the construction.
Deaths due to the collapse of wooden houses amounted to nearly 5,000. However, the number of collapsed wooden houses was about 60,000. So, on average, every 12 collapsed wooden houses killed one person.

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A Comparison of the Seismic Destructiveness of Recent Earthquakes

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SUMMARY

In this study a set of accelerograms recorded during recent earthquakes experienced in four different countries is analyzed. Results using these accelerograms and several parameteres proposed in the literature for measuring seismic destructiveness are compared with global building damage observed during the earthquakes studied.

INTRODUCTION

Several earthquakes in the last decade have shown that there is an urgent need of understanding the nature of seismic destructiveness. In this paper a set of accelerograms recorded during earthquakes experienced in four different countries is analyzed. Results using these accelerograms and several parameteres proposed in the literature for measuring seismic destructiveness are compared with building damage observed during the earthquakes studied.

EARTHQUAKE GROUND MOTIONS

A set of four earthquake records was selected for this study: a) the SCT-EW record (SCT) obtained in Mexico City during the September 19, 1985 earthquake; b) the Sylmar record (SYL) obtained in Northridge, California, during the January 17, 1994 earthquake; c) the Kobe record (KOB) obtained in the Kobe Marine JMA Observatory during the January 17, 1995 earthquake; and d) the Viña del Mar record (VM) obtained in the city of Viña del Mar during the March 3, 1985 Chile earthquake. In the 1985 Mexico City earthquake most building damage and collapses were observed in frame systems, in the range of 6-12 floors (Rodriguez, 1994). On the contrary, during the 1985 Chile earthquake buildings in Viña del Mar showed in general a good seismic behavior, mainly because typical of chilean building construction practice is the use of relatively stiff buildings having structural walls for lateral load resistance. Typical building damage during the 1995 Kobe earthquake has been well documented in the literature (Muguruma et al, 1995). It is of interest that evidences of severe damage or collapses were observed mainly in buildings lower than 10 stories.

PARAMETERS FOR MEASURING SEISMIC DESTRUCTIVENESS

A parameter for measuring seismic destructiveness has been proposed by the first author and is used in this study. The reader is referred elsewhere (Rodriguez, 1994) for a detailed description of the method of analysis and assumed hypotheses for deriving the proposed parameter, I_D, which is defined as:

$$I_D = N_e \left(\frac{D_{rm}}{D_{rd}}\right)^2 \tag{1}$$

The parameter N_e results of normalizing the total hysteretic energy per unit mass dissipated by a SDOF system. The parameter D_{rm} is the maximum roof drift ratio in a multistory building and is defined as:

$$D_{rm} = \frac{\delta_m}{H} \tag{2}$$

where δ_m is the maximum roof displacement and H is the height of the building. By using several building parameters, an approximate expression for D_{rm} has been proposed by (Rodriguez, 1994). Among these parameters, λ relates the fundamental period of the building, T^*, with the number of floors n according to the following expresion:

$$T^* = \frac{n}{\lambda} \tag{3}$$

Since the parameter I_D is a global measure of earthquake damage, in this study the parameter λ is taken equal to 14.1 for structural wall buildings and 7.1 for frame and dual systems. These values consider the fact that effective fundamental periods of buildings measured with actual earthquake records are significantly longer than initial periods. Lower values for λ should be used for RC buildings in the lake bed area of Mexico City, which is caused by base rotation due to soil flexibility. In this case, a value of about $1.3T^*$ has been suggested (Bazan et al, 1992) for evaluating the soil-structure-interaction period, where T^* is evaluated considering the fixed-base case. The parameter D_{rd} is a measure of an acceptable level of roof displacement ratio during an earthquake and in all cases analyzed in this study was taken equal to 0.01 (Rodriguez, 1994).

For the sake of simplicity and according to a review of typical building construction practice in countries corresponding to the earthquakes studied, structural wall buildings were considered when using the proposed parameter for analyzing the 1995 Kobe earthquake and the 1985 Chile earthquake. In the other hand, frame or dual systems were considered for analyzing the 1985 Mexico City earthquake and the 1994 Northridge earthquake.

Results of the evaluation of I_D for given maximum global displacement ductility ratios, μ_m, assuming elastoplastic behavior, and considering a fraction of critical damping, ξ, equal to 0.05, are reported by Rodriguez and Aristizabal (1996). The results show that the earthquakes with the highest seismic destructiveness are asssociated to the SCT and SYL records. The lower seismic destructiveness corresponds to the KOB and VM records. Caution should be taken when interpreting these results, since an important property of the parameter I_D should be considered. An inspection of (2) shows that I_D is directly proportional to D_{rm} squared. According to this property and assumed λ values, when analyzing the KOB record for more flexible structures, such as frame or dual systems, I_D values four times those considering structural wall buildings should be expected This property might help to understand the large amount of structural damage or collapses in flexible structures, such as frame systems, observed during the 1985 Mexico City and 1995 Kobe earthquakes.

A parameter for measuring seismic destructiveness, P_D, was proposed by Araya and Saragoni (1985). This parameter considers the Arias intensity I_A (Arias, 1970) and the intensity of zero crossings ν_0. Another parameter for measuring seismic destructiveness is the response spectrum intensity (S_I), which was proposed by Housner (1952), and uses the linear elastic pseudo-velocity, S_V. The PGA, referred as A_{max}, is also analyzed as a possible measure of earthquake intensity.

Normalized parameters I_D, P_D , S_I and A_{max} for the four earthquake records are shown in Fig 1. Each parameter was normalized with respect to its maximum value. Results of the evaluation of I_D in Fig 1 correspond to ξ and μ_m values equal to 0.05 and 4, respectively. In these results the earthquake records are ordered according to their maximum I_D values. Results of Fig. 1 show that according to the parameters I_D and P_D the 1985 Mexico City earthquake has the highest destructiveness, which is in agreement with observed building damage in the earthquakes studied.

Results of Fig 1 also show that according to the parameter S_I the SYL record has the highest seismic destructiveness, followed by the KOB and SCT records, with comparable values of seismic destructiveness. A comparison of these results and observed building damage during the earthquakes related to these records indicates that S_I is not a reliable parameter for measuring seismic destructiveness. In addition, Fig 1 shows that according to I_A the KOB record has higher intensity than the corresponding to the SCT record, and is comparable to the intensity of the VM record, which again is not consistent with observed building damage in the earthquakes related to these records. Results of this type are also obtained when using A_{max} (Fig 1).

CONCLUSIONS

Results for the evaluation of the proposed parameter using data from four earthquakes show an acceptable correlation with damage observed during the earthquakes studied. According to the proposed parameter, the seismic destructiveness corresponding to the 1985 Mexico City earthquake is higher than those calculated for the other earthquakes. However, higher calculated seismic destructiveness should be expected for the 1995 Kobe earthquake if more flexible structures are considered, such as frame or dual systems. This point out the importance of using structural wall buildings for reducing damage during earthquakes.

Results using the proposed parameter were also compared with those for other parameters proposed in the literature. It was found that results using Housner intensity, Arias intensity and PGA have a poor correlation with observed earthquake damage in the earthquakes studied.

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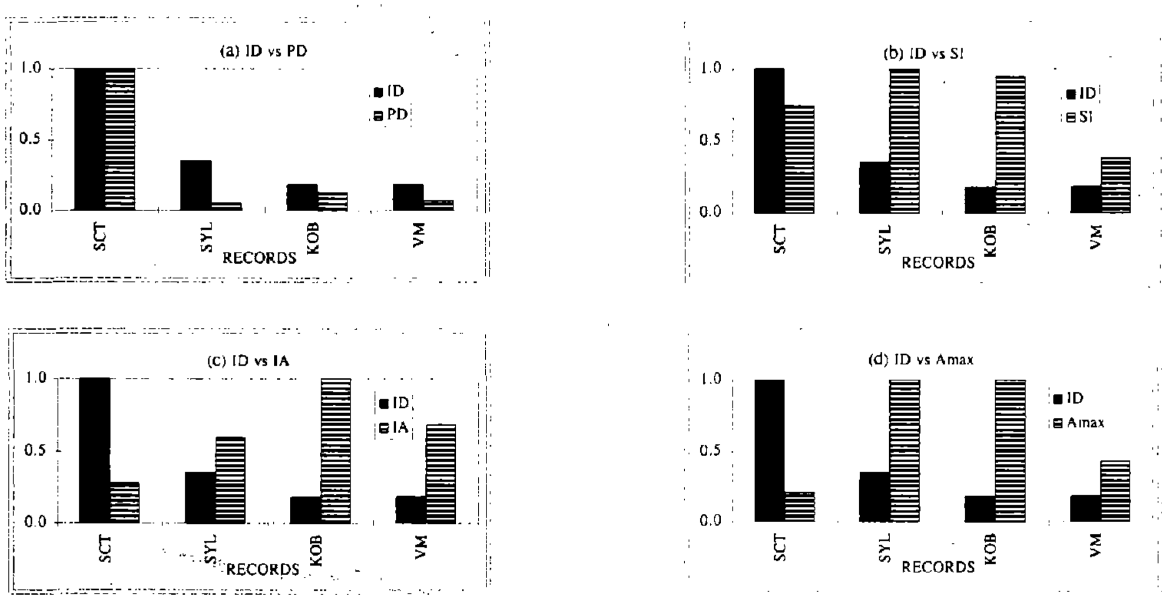

Fig.1 Normalized parameters I_D, P_D, S_I and A_{max}

Earthquake Damage Mitigation of R.C. Frames by Using Damped Steel Braces

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SUMMARY

The conventional aseismic design is based on the use of the inelastic design spectrum obtained through a reduction of a smoothed elastic response spectrum according to the average ductility level accepted for the structure. A novel technique aims to reduce the ductility demand of framed structures by the insertion of a damped steel bracing. This can lead to use light additional design and detailing rules for the frame members (not so conservative as those adopted in the conventional design approach); on the other hand, these rules often turn out to be not satisfied for existing framed building to be retrofitted by using damped bracing. Therefore, whether inelastic deformations of the framed structure occur during severe earthquakes, the mechanical degradation can become important and the inelastic design spectrum should account for it. A suitable *damage index* more conservative than ductility factor should be adopted for a reliable design.

The main purpose of this paper is the development of a simplified yet reliable procedure for a practical seismic design of the damped steel bracing to attain a designated protection level (corresponding to an accepted damage level) for a reinforced concrete (r.c.) framed structure. To check the effectiveness and reliability of the design procedure, a numerical investigation is carried out comparing the nonlinear seismic responses of unbraced and damped braced r.c. frames. Results obtained by adopting different *damage indexes* prove the efficiency of the damped braces designed according to the proposed design criteria, even for a brace stiffness relatively low.

DESIGN CRITERIA OF THE DAMPED BRACING AND DAMAGE INDEXES

Several kinds of damped bracing systems were proposed which basically differ for typology and energy-dissipation mechanism (friction, hysteresis, viscosity or viscoelasticity). A general discussion about experiences in different countries can be found, e.g., in the papers by Aiken I.D. & Whittaker / Ciampi / Kitamura & al. / Martinez-Romero E., 1993. For sake of clearness, in this paper the attention is focused on the friction-damped bracing proposed by Pall & Marsh (1982), which makes use of the device in **Figure 1A**. The loads in the phase in which slippage occurs are shown in **Figure 1B** (N_g=global slip load in the tension brace; N_l=local slip load in each slip joint; N_{cr}=buckling load of the compression brace, practically negligible).

Supposing the properties of the framed structure are known and assuming a horizontal-load pattern (e.g., that corresponding to the first mode shape of the braced frame supposed elastic), the damped bracing system is designed according to criteria analogous to those already adopted in a previous paper (Vulcano, 1994) and summarized below:

- the elastic-stiffness distribution of the braces is similar to that of the bare frame, that is, the same value of the stiffness ratio $K^*=K_b/K_f$ is assumed at each storey between the two lateral elastic stiffnesses

provided, respectively, by the bracing system (assumed elastic) and the bare frame;
- the distribution law of the global slip load N_g in the damping devices at different stories is similar to that of the elastic axial force induced by the lateral loads in the tension braces before the slippage, so that the totally dissipated energy be as large as possible;
- the selection of the N_g value at the generic storey is restricted to the range (N_{min}, N_{max}), where the lower bound, reasonably assumed as N_{min}=0.5N_{max}, should ensure that the device does not slip under normal service loads and moderate earthquakes (or wind), whereas the upper bound should avoid any yielding of the frame members before slippage in the device as well as the occurrence of undesirable phenomena in the frame columns (e.g., buckling, brittle failure in r.c. columns, etc.);
- due to the last two assumptions, the slip load can be characterized at each storey by the same value of the slip-load ratio $N^*=N_g/N_{max}$;
- the optimum slip load is selected by a criterion of minimization with reference to some index intending to evaluate the efficiency of the damped bracing system as the ratio between the values assumed by a suitable parameter representative of the frame damage for the damped braced frame (BF) and the unbraced frame (UF).

To evaluate the efficiency of the damped braces for reducing the frame damage, different *performance indexes* were proposed (Filiatrault & Cherry, 1988; Vulcano, 1994). In this paper, *damage indexes* , which are based on *damage functionals* available in the literature (for a general discussion see, e.g., paper by Cosenza & Manfredi, 1992), are considered. A *damage index* is defined as the ratio

$$DPI = \frac{D_m^{(BF)}}{D_m^{(UF)}} \tag{1}$$

where $D_m^{(..)}$, representing the level of the frame damage, is calculated as weighted average of the values assumed by a suitable *normalized damage functional* D_μ in the critical sections of the frame members. Exactly, D_μ is calculated, with reference to an elastic-perfectly plastic moment-curvature law, as

$$D_\mu = \frac{\mu - 1}{\mu_{u,mon} - 1} \qquad \text{or, in alternative,} \qquad D_\mu = \frac{\mu_F}{\mu_{Fu,mon}} \tag{2a;b}$$

according to whether the local damage is evaluated with reference to a *ductility factor* μ (e.g., *kinematic* , *cyclic* or *hysteretic*) or to a suitable *damage functional* μ_F (e.g., *Park & Ang*, *Banon & Veneziano* or *plastic fatigue by Banon et al.*). $\mu_{u,mon}$ and $\mu_{Fu,mon}$ represent the ultimate values which can be attained, respectively by μ and μ_F, in a monotonic test.

NUMERICAL RESULTS AND CONCLUDING REMARKS

A numerical investigation has been carried out with reference to the three- and five-storey test structures schematically shown in **Figure 2**, which consist of r.c. frames with friction-damped steel braces (see **Figure 1**). The r.c. frames have been designed, according to C.E.B. Seismic Code (1987), as "weak beam-strong column" structures in a high-risk seismic zone, assuming soil profile S_2. The friction-damped bracing system has been designed according to the criteria illustrated above. The main characteristics (member size and ultimate moments, etc.) of the r.c. frames and more detail on the design of the test structures can be found in a paper by the authors (1995).

Many time-step dynamic analyses have been carried out using the procedure already adopted in the paper by Rega & al., 1990. Elastic-perfectly plastic models are adopted to simulate the hysteretic behaviour of the frame members and that of the friction-damped bracing at each storey, while an implicit two-parameter time integration scheme and an initial stress-like iterative procedure are used to perform the seismic nonlinear analysis. Three artificial motions matching the C.E.B. response spectrum for soil profile S_2 have been considered, assuming a peak ground acceleration A=0.35g. All the results have been obtained as an average of those corresponding to these motions.

For sake of brevity, only the results in **Figure 3** are presented: *damage indexes* for the framed part of three-storey test structures are plotted versus the slip-load ratio N* with reference to two values of the stiffness ratio K* (0.5 and 2.0, which can be considered relatively low and high, respectively). It is interesting to note that the scattering of the results when referring to different damage parameters is greater for K*=0.5. Generally, the *Banon & Veneziano index* (BVI) is the larger one, while the *plastic fatigue index* (PFI) is the the smaller one. This last result can be explained noting that PFI (so as the *hysteretic ductility index* HDI, which presents values lightly larger than PFI) accounts exclusively for hysteretic effects, relatively more marked because of the stiffening due to the insertion of braces producing a number of loading cycles greater than that for the unbraced frame.

According to the results in **Figure 3** and others omitted for sake of brevity it can be concluded that:
- the damped braces, even with a relatively low stiffness, prove to be very effective for mitigating, during a strong ground motion, the average and local damages of the framed structure;
- the efficiency of the damped bracing is greater for larger values of K*, even though a relatively low value of the stiffness ratio (e.g., K*=0.5) produces a noticeable protection of the frame;
- the curves representing the *damage indexes* versus the slip load ratio N* show a similar shape, with a steep decrease for relatively low values of N* and a rather stable trend for higher values of N*;
- this trend is favourable when the device should not slip during a severe earthquake because of accidental factors (e.g., imperfection in tuning the slip load of the device, change of the environmental conditions influencing the behaviour of the devices, etc.);
- a larger stiffness of the damped bracing leads to a wider amplitude of the practically stable branch of the above curves, that is to a wider range in which the N* value can be selected assuring a greater protection level of the frame.

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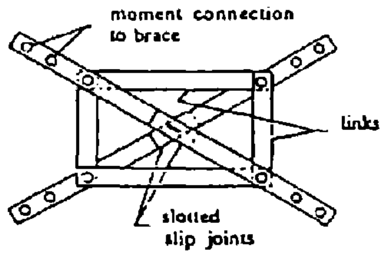
FIGURE 1
FRICTION-DAMPED BRACING:
(A) DEVICE
(B) FORCES UNDER SLIPPAGE



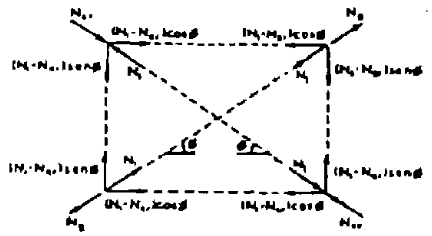
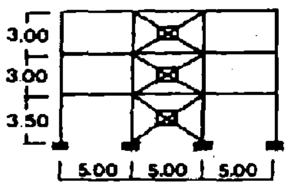


FIGURE 2
TEST STRUCTURES



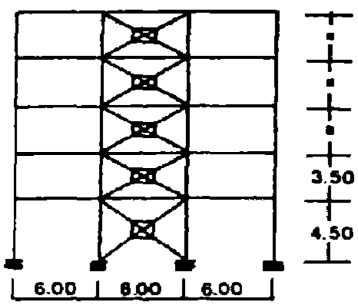
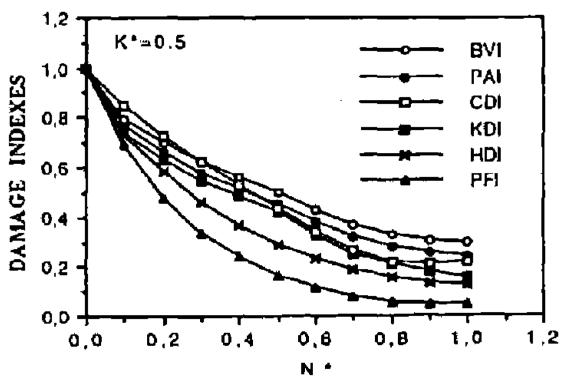
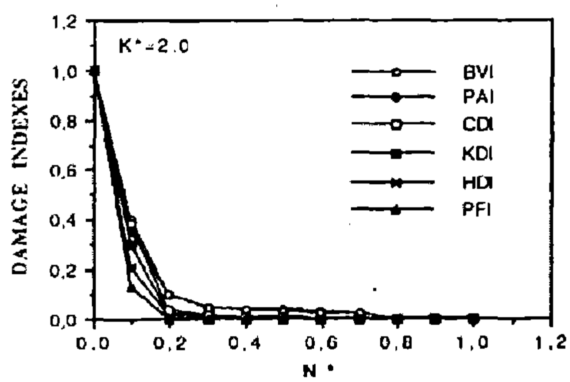


FIGURE 3
DAMAGE INDEXES VS. SLIP-LOAD RATIO FOR THREE-STOREY TEST STRUCTURES

DAMAGE INDEXES:

KDI: Kinematic ductility	PAI: Park & Ang
CDI: Cyclic ductility	BVI: Banon & Veneziano
HDI: Hysteretic ductility	PFI: Plastic fatigue





Cross-Modal Estimators in Spectral Superposition

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SUMMARY

The problem of signs of Cross-Modal Estimators that appear in several applications of Spectral Modal Superposition is addressed. It is shown that the signs are not ambiguous, notwithstanding the alternating character of the motion. This is actually a property of the structural dynamic system. The discussion led to a formula for the maximum value of a response variable in a two-degree of freedom system that can be regarded as a deterministic justification of double-sum estimation schemes.

INTRODUCTION

The apparent inability of Spectral Modal Superposition (SMS) to provide information needed for design, which in static or time-history analyses is obtained from compatibility or equilibrium, can be removed by procedures (Vásquez, 1992) which introduce a Cross-Modal Estimator (CME) defined as

$$\overline{RS} = \sum_{i,j} \rho_{ij}\, r_i s_j$$

for every pair of response variables, $r(t)$ and $s(t)$, involved in the corresponding compatibility or equilibrium relationship. Typically, through these relationships a given response variable is defined as a linear combination $u(t) = \alpha r(t) + \beta s(t)$, and its estimator of maximum value is found to be given by

$$U^2 = \alpha^2 R^2 + 2\alpha\beta\, \overline{RS} + \beta^2 S^2$$

CMEs also solve the problem of finding, for earthquake excitation of arbitrary direction, the directional maximum of a response variable. For a one-component excitation, the estimate is (Vásquez., 1987)

$$R^2 = \tfrac{1}{2}\left(R_x^2 + R_y^2 + \sqrt{(R_x^2 - R_y^2)^2 + 4\,\overline{R_x R_y}} \right) \qquad \text{where} \qquad \overline{R_x R_y} = \sum_{i,j} \mathcal{L}_{xi} \mathcal{L}_{yj} \rho_{ij}\, \bar{r}_i \bar{r}_j$$

The CME is expressed in terms of modal participation factors for excitation in two orthogonal directions. For a two-directional excitation, an extension of this formula can be used (Vásquez , 1996).

Finally, an important problem in which CMEs appear is finding the value that a given response variable has at the same time t_s at which a second variable attains its maximum value. This is the synchronous value of the first variable with respect to the maximum value of the second one. The synchronous value of $r(t)$ with respect to the maximum estimation S of $s(t)$, is given by (Gupta, 1990, Vásquez, 1992)

$$Sr(t_s) = \overline{RS} = Rs(t_r)$$

The signs of two response variables, when one of them is at its maximum, are thus either equal or opposite, according to whether their CME is either positive or negative (Vásquez, 1992). Therefore, if there is no ambiguity with respect to the signs of CMEs, the somewhat widespread notion that SMS "looses the signs" proves to be misconception. In a certain way signs, and more in general, linear combinations, are encapsulated in the CMEs.

SIGNS OF CROSS-MODAL ESTIMATORS

Certainly, CMEs can be either positive or negative. However, the supposedly sign-suppressing character of SMS techniques, has led people to raise doubts about the significance of such signs. The argument is that signs that appear in CMEs are subjected to alternating reversals, so they do not seem to be clearly defined; they can be regarded as ambiguous. This is, of course, a serious problem that needs to be solved, before expressions involving CMEs can be properly used.

The first possible source of ambiguity is found in the choice of a sign in the normalization of the modal shapes. However that choice does not really introduce any uncertainty in the analysis. Actually, changes of sign in modal shapes are utterly immaterial, as in fact is the case in all modal superposition applications. Under a sign reversal of a modal shape, the corresponding excitation factor, $\mathcal{L}_i$, and the modal value of the response variable under consideration, $\bar{r}_i$, change signs simultaneously. The product of the excitation factor times the modal shape response value, i.,e., $r_i = \mathcal{L}_i \bar{r}_i$, is an invariant property of the system, that has a very specific sign. The arbitrary choice of sign involved in the normalization process of any given mode is then of no significance.

The second source of ambiguity is the use of spectral readings which, for each frequency, represent the absolute value of the maximum obtained from the time-history response of the corresponding single degree of freedom system. However, the possible inconsistency is not inherent to the definition of CMEs. Rather, if indeed a problem, it would be one related to double-sum spectral superposition schemes. In fact, when the superposition formula being used is that of the SRSS method, each term in the then single-index sum is associated with one and the same mode. Maximum modal coordinates then occur under quadratic, sign insensitive, forms. Therefore, at least in such a case, the signs of those maximum coordinates are definitely irrelevant.

It is when using a double-sum estimation formula that the loss of the signs of the maximum modal coordinates seems to be questionable. Actually, the problem is already present when the double-sum formula is used for a standard estimation of the maximum of a single response value.

If that case is made clear, any ambiguity in the signs of CMEs also disappears. That is what is going to be discussed in the analysis that follows. The discussion will be based in the fact that double-sum estimation formulas are designed to take into account coupling effects occurring when modal frequencies are very closely spaced. Modes with widely spaced frequencies are not coupled. This is reflected in ρ_{ij}'s that are zero, or very near to zero. The part of the double-sum associated with the largely uncoupled modes contains essentially the same sign insensitive terms of the SRSS. Therefore, the terms originating in the closely coupled modes are then the only ones that need to be considered.

A time-history modal superposition analysis serves the purpose of establishing the form that the coupled terms of the double-sum must have. And since coupling is a two-variable phenomenon, a two-degree of freedom system has sufficient generality, and is suitable for the discussion. In such a two-degree of freedom system, a given response variable $r(t)$ will have a time-history response that can be expressed in terms of the modal coordinates as $r(t) = \zeta_1(t)r_1 + \zeta_2(t)r_2$. If the signs of r_1 and r_2 are equal, its maximum value will occur at an instant of time t_r within the supposedly short interval of time starting at the occurrence of the maximum of the first modal coordinate, t_1, and ending at the occurrence of the maximum of the second modal coordinate t_2. Furthermore, the maxima of both modal coordinates ζ_{1max} and ζ_{2max} should differ only slightly in value. And certainly the two must be of equal signs. During the interval of time t_1, t_2 and its vicinity, the two normal coordinates can be reasonably approximated as

$$\zeta_1(t) = \zeta_{1max} \cos\omega(t - t_1) \qquad\qquad \zeta_2(t) = \zeta_{2max} \cos\omega(t_2 - t)$$

In these expressions ω is the average value of the two almost identical modal frequencies. By substituting from these approximations, the response variable can be written as

$$r(t) = \zeta_{1max} \cos\omega(t - t_1)\, r_1 + \zeta_{2max} \cos\omega(t_2 - t)\, r_2$$

And through direct differentiation, its velocity is readily found to be

$$\dot{r}(t) = -\omega\, \zeta_{1max} \sin\omega(t - t_1)\, r_1 + \omega\, \zeta_{2max} \sin\omega(t_2 - t)\, r_2$$

The form of these two equations suggests summing the squares of both the variable and its derivative divided by ω, so as to obtain, by using simple trigonometric identities, a relationship in which some of the sines and cosines cancel or simplify. The result is most rewarding. In fact, the intermediate step

$$\begin{aligned} r^2(t) + \frac{\dot{r}^2(t)}{\omega^2} = {} & (\zeta_{1max}\, r_1)^2\left(\cos^2\omega(t - t_1) + \sin^2\omega(t - t_1)\right) + (\zeta_{2max}\, r_2)^2\left(\cos^2\omega(t_2 - t) + \sin^2\omega(t_2 - t)\right) \\ & + 2(\zeta_{1max}\, r_1)(\zeta_{2max}\, r_2)\left(\cos\omega(t - t_1)\cos\omega(t_2 - t) - \sin\omega(t - t_1)\sin\omega(t_2 - t)\right) \end{aligned}$$

leads to the very compact quadratic expression

$$r^2(t) + \frac{\dot{r}^2(t)}{\omega^2} = (\zeta_{1max}\, r_1)^2 + (\zeta_{2max}\, r_2)^2 + 2(\zeta_{1max}\, r_1)(\zeta_{2max}\, r_2)\cos\omega(t_2 - t_1)$$

which presents the rather surprising property of having a time independent right hand side. Of course, the maximum response, $r(t_r)$, occurs at instant of time at which the velocity, $\dot{r}(t)$, is zero. Hence, the right hand side, invariant for all t, is then bound to be equal to the square of the required maximum response. The formula for the maximum value is then

$$r_{max}^2 = (\zeta_{1max} r_1)^2 + (\zeta_{2max} r_2)^2 + 2(\zeta_{1max} r_1)(\zeta_{2max} r_2)\cos\omega(t_2 - t_1)$$

If the signs of r_1 and r_2 are opposite, the maximum r_{max} given by this approximation will occur at an instant of time t_r outside the interval t_1, t_2. Actually, t_r can be found to be equal to, approximately, the weighted average of t_1 and t_2 with weights r_1 and r_2. If t_r is still in the vicinity of the interval t_1, t_2, the approximation will continue to hold. However, if the absolute values of r_1 and r_2 approach each other, t_r will tend to infinity while the value of r_{max} will obviously tend to zero. Of course, this is a situation of tuning. If the response of the modal coordinates were actually sinusoidal, as in a dampless free vibration, it would eventually account for the reinforcement of the two harmonic waves. If as in earthquake response, that is not the case, the two waves corresponding to the modal coordinate responses will cancel each other, except for small differences of no practical significance at the level of design controlling maxima, and the approximation for r_{max} can be considered to continue to be applicable.

The expression for r_{max} shows that double-sum estimators are indeed suitable for cases of nearly coincident frequencies. Actually, it can be regarded as a deterministic justification of the pertinence of double-sum estimation schemes for cases in which modal coupling is important. The coupling coefficient, which should be equaled to ρ_{ij}, is found to be $\cos\omega(t_2 - t_1)$. Since it is the cosine of a small angle, it has a positive value. The modal coordinate maxima, ζ_{1max} and ζ_{2max}, do have the same sign, so their product is always positive. The actual sign is then immaterial, and it can be arbitrarily taken as positive, as is the case in readings from a spectral chart. The signs of the modal excitation factor, $\mathcal{L}_i$, and those of the modal shape value of the response variable, r_i, must, of course, be retained. This proves that there is no ambiguity in the signs of modal coupling terms in double-sum estimators. It further implies that the signs of CMEs can be regarded as system-defined properties.

CONCLUSIONS

The signs of CMEs that appear in procedures allowing the introduction of compatibility and equilibrium relationships in a SMS context were shown to be well-defined properties of the structural dynamic system. Doubts at this respect, raised from misconceptions as to a supposedly "sign suppressing" characteristic of spectral techniques were thus removed. It was found that in all cross-modal terms spectral ordinates must be read as positive; participation factors and modal shape responses must retain their signs. These results were obtained from the derivation of a formula establishing the maximum of a response variable considering two closely coupled modes. That formula can also be regarded as a deterministic justification of the pertinence of double-sum estimation schemes.

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Design Parameters Optimization for Reinforced Concrete Structures Incorporating Energy Dissipation Bracings

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SUMMARY

This paper briefly summarizes the analytical and the experimental results of a research dealing with the earthquake protection of reinforced concrete (R.C.) moment resisting frames (MRF) obtainable with the application of steel bracings incorporating energy dissipation friction devices. This work is intended to contribute to the discussion about the problem of defining the proper design parameters for such dual systems, traditionally referred to as K*, e.g. the ratio between the brace stiffness and the lateral stiffness of the MRF, and R*, e.g. the ratio between the slip-load and the MRF's lateral ultimate strength.

STRUCTURAL MODELING AND NUMERICAL ANALYSIS

Relying on previous works, which have demonstrated the allowance and the effectiveness of representing complex multi-story structural systems via simpler single-story models, a parameter study has been carried out upon a numerical model using the ANSR computer program. As is shown in **Figure 1**, the model assimilates the space structure to a three-degrees-of-freedom oscillator provided with five inelastic springs. Two of these springs act like the R.C. plane frames in the direction of the earthquake forces, two springs simulate the bracings incorporating the friction devices and, eventually, the fifth spring acts like the R.C. plane frames in the direction orthogonal to the seismic excitation.

In the preliminary sensitivity analysis, the lateral period of vibration, equal to 0.8 sec, obtained by applying a weight of about 4000 kN to the center of gravity, was maintained constant, whereas K* was varied from 0 to 9 and R* was increased from 0 to 1. The non-linear dynamic lateral response of such a dual system has been evaluated considering the following parameters: 1) the ratio E_d/E_i between the energy dissipated by the hysteretic behavior of the friction devices and the input energy, both calculated at the end of the seismic event, 2) the ratio δ^*_{max} between the maximum lateral displacement of the friction braced MRF (FBMRF) and that of the bare MRF subject to the same earthquake excitation, and 3) the maximum absolute displacement of the FBMRF, δ_{max}. The computer model was then subjected to numerous artificial as well as real accelerograms, all scaled to 0.25 g, capable of supplying the structure with input energies up to 100 kJoules. Under these conditions, the model exibited the best response, in terms of both energy dissipation and minimization of lateral displacement, for a value of R* equal to about 0.8.

MODEL OPTIMIZATION BY EXPERIMENTAL TESTING

Upon completion of the preliminary sensitivity analysis, the numerical procedure has been checked and optimized through the comparison between the experimental response obtained by testing the real-scale 3D single-story R.C. frame shown in **Figure 2** and the response anticipated by the numerical model of the same frame. The R.C. frame has a square plan and is composed of four columns and as many girders bearing a solid slab. The structure was designed according to the Eurocodes (EC2 and EC8). The dissipative chevron-type steel bracings are incorporated into two opposite bays in the direction of the excitation. A couple of friction pads were glued at both sides of a gusset bolted to the braces. Two steel angles, connected to the middle section of the girders, were tightened to the gusset by means of three high-strength bolts in order to apply as uniform a pressure as possible on the pads. The bolts were instrumented with strain-gages in order to control the initial transverse force applied to the friction pads. The energy dissipation devices are activated when a relative displacement between the gusset and the girder is attained, that is when the story displacement exceeds the elastic displacements of the bracings. The efficiency of the friction devices, as regards the stability of the hysteretic behavior and the slip-load repeatability, was previously tested on a single component, under severe cyclic test conditions.

The lateral period of vibration of the braced frame, whose total weight was about 108 kN, was equal to 12.19 Hz. The K* ratio resulted equal to about 9. The structure was excited in a sine-sweep mode in the frequency range from 1 Hz to 18 Hz by means of a servo-hydraulic shaker acting in the floor middle plane. The lateral force amplitude was varied from 1470 N to 5890 N. The maximum input energy was of the order of 1 kJoule. Under these test conditions the optimum experimental response occurred for a value of R* equal to 0.2.

As mentioned above, a structural identification procedure, based upon the actual dynamic characteristics determined experimentally, has allowed for the formulation of the optimized ANSR numerical model of the mock-up, based on the analytical model depicted in Figure 1. The structural response obtained with the ANSR model was in a good agreement with the experimental results.

A further sensitivity analysis has then been carried out upon the two models (the former simulating a R.C. structure in the real world and the latter acting like the lab mock-up) aimed at evaluating the influence on the optimum value of R* of the following parameters: period of vibration, type of excitation (either sine force applied to the center of gravity or earthquake accelerograms acting at the base), amount of input energy (under comparable durations of excitation). The results of the sensitivity analysis are as follows:

1. The period of vibration has a negligible influence on the optimum R* curves (see **Figure 3**) whereas the excitation level does have a significant influence (see **Figures 4 and 5**). These characteristics of structural response are similar for both the earthquake accelerograms and the sine excitations.
2. Values of R* from 0.4 to 0.6 represent a good trade-off to resisting different earthquake force amplitudes with a satisfactory energy dissipation (in excess of 60% of the input energy).
3. By using sine excitations it has been observed that, augmenting the structure weight, higher optimum values of R* are attainable for lower levels of excitation (see **Figures 6 and 7**).

LAB TESTS ON A MODIFIED MOCK-UP

In order to investigate how the structural behavior of the FBMRF might possibly change if either the weight or the excitation level had been augmented, a further experimental campaign has been carried out on a modified version of the mock-up. The weight was increased up to about 200 kN by means of supplemental masses, so as to lower the lateral period of vibration to 9.1 Hz. The modified structure was then shaken in a sine-sweep mode in the frequency range stretching from 5 Hz to 11 Hz. The force amplitude was increased up to 7.5 kN.

The results of the experimental campaign are shown in **Figures 8, 9 and 10** In essence, the following remarks can be drawn:

1. The input energy E_i increases with R*. The dissipated energy E_d is greater for R* values between 0.4 and 0.6.
2. The E_d/E_i *vs* R* curves exibit a maximum at R* equal to 0.4 with a minimum at R* equal to 0.8 This tendency was previously shown by the numerical analysis: in fact, by increasing the excitation amplitude either the optimum R* moves to higher values or, at least, the E_d/E_i *vs* R* curves become asymptotic
3. By increasing the R* values, the maximum displacement δ_{max} increases. This tendency, that is coherent with the E_d/E_i *vs* R* curves, demonstrates that the optimum response is obtained with low R*'s. The fact that the optimum ratio E_d/E_i is attained for R* equal to 0.4 and the minimum δ_{max} occurs at R* equal to 0.2 validates the results of the numerical analysis: the tendency of the minimum δ_{max} to occurr at higher values of R* is less remarkable than it happens to the E_d/E_i ratio.
4. The numerical model proves effective in anticipating the structural behavior of the FBMRF in terms of input energy. The numerical model fails to furnish the correct amount of energy dissipated by friction, even though it is capable of tracing the trend qualitatively The fact that the numerical model gives a larger amount of dissipated energy than it has been experimentally determined might be due to the mathematical description of the lateral displacement vs lateral force relationship which, in the real dual system, is by far more complex than the usual bi-linear constitutive law used in the ANSR program to describe the kinematic behavior of the braced frame
5. The numerical results are in a fairly good agreement with the experimental results as far as the peak story displacements are concerned. However, the computer model gives smaller displacements than it occurs in the mock-up. It is apparent that it is a consequence of the larger energy dissipation accounted for by the ANSR model.

RESEARCH IN PROGRESS

The above results are relevant and apply to mass-, stiffness- and strength-symmetric structures. In the lab mock-up the energy dissipation devices were activated by the same slip-loads and were mounted atop two bracings having the same lateral stiffnesses. A further numerical campaign has then been initiated, using the ANSR model, to investigate the behavior of asymmetric structures as well.

Two values of weight (i.e. 108 kN and 216 kN) and two values of brace stiffnesses (K1 equal to 26,000 kN/m and K2 equal to 68,000 kN/m) have been considered in the analysis. Several combinations of these values have been taken leading to ten different arrangements. For either values of the weight the following structural models have been analyzed: a) R.C. frame with only one type-K_1 bracing; b) R.C. frame with only one type-K_2 bracing; c) R.C. frame with two type-K_1 bracings one of which incorporating a jammed friction device; d) R.C frame with two type-K_2 bracings one of which incorporating a jammed friction device and e) R.C. frame with a type-K_1 bracing in one bay and a type-K_2 bracing in the other.

All these arrangements have been investigated under sine excitations with force amplitudes varying from 24 kN to 283 kN. The results of the numerical analysis can be summarized as follows:

1. The energy dissipated is always in excess of 80% of the input energy.
2. The difference between the displacements of the two plane frames, where the bracings are incorporated in, is small, particularly for R* values in the range from 0.4 to 0.6.
3. If the two bracings, even with different stiffnesses, are designed to have the same slip-loads, the difference between the lateral displacements of the two frames, where the bracings are incorporated in, is small.

More experimental activity will be devoted in the next future to validate the above numerical results.

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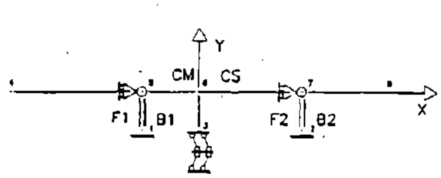


Figure 1

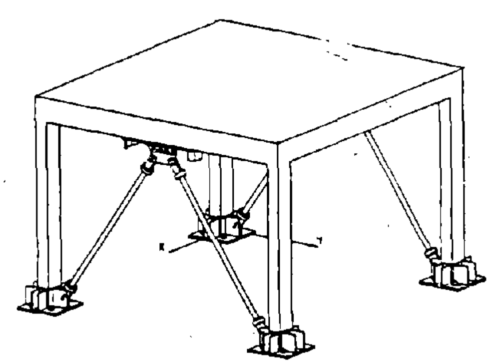


Figure 2

figure 3

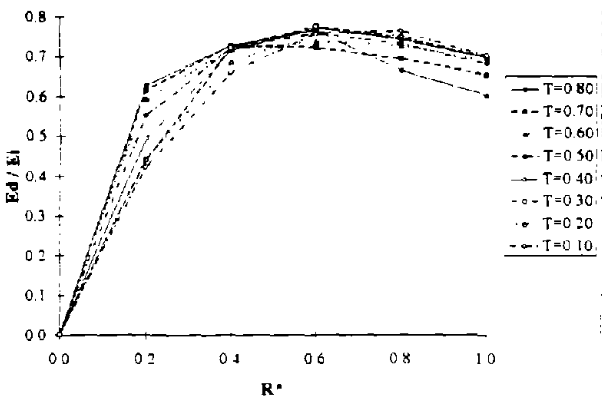
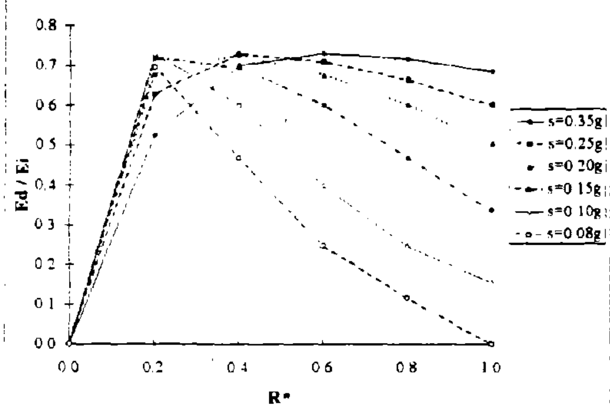


figure 4



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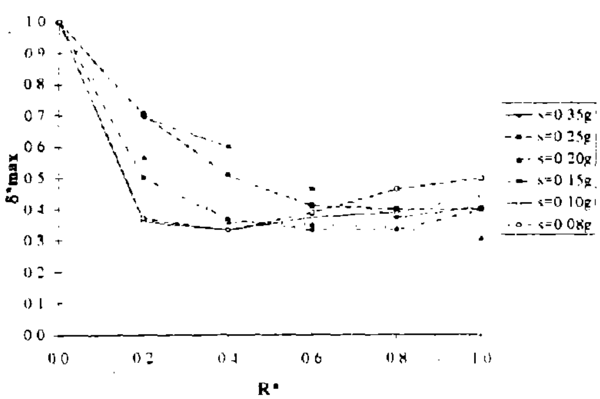


figure 6

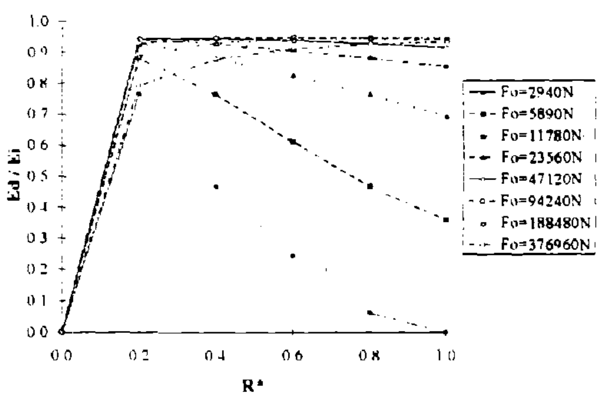


figure 7 (M=21640 kg)

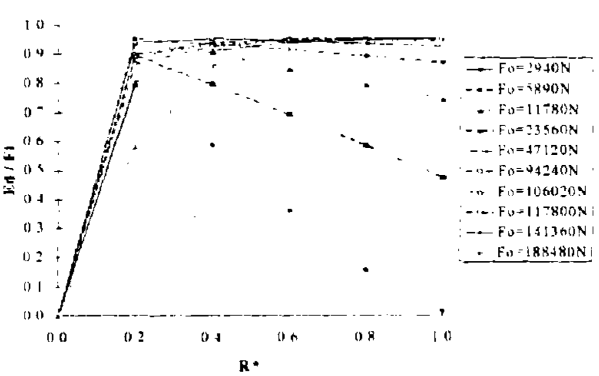


figure 8

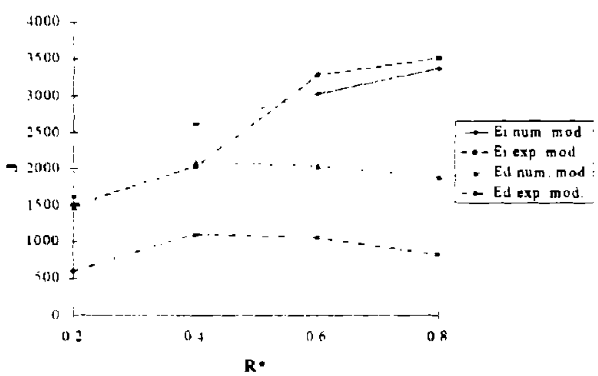


figure 9

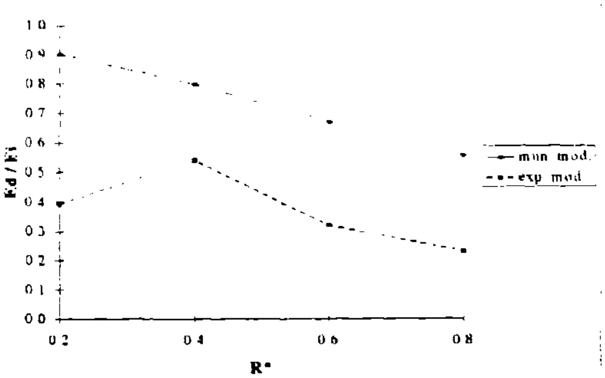
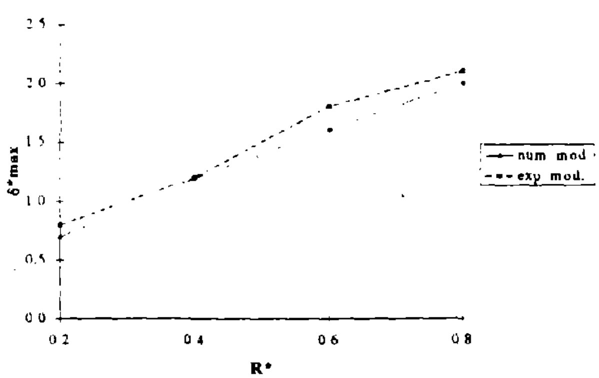


figure 10



Proposed Linear Elastic Response Spectras Based on Frequency Content Comparisons Between Moderate Earthquake and Ambiance Vibration Records

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SUMMARY

Linear-elastic design response spectras for a city are obtained through a comparisson between ambiance vibrations and moderate earthquake records frequency contents. These comparissons are performed calculating cross spectral densities for ambience vibration as well as for moderate earthquake records.

INTRODUCTION

There are several cases of important soil effects during earthquakes. In fact, the Fourier analysis of strong motions like those of the 1985 Mexico earthquake have clearly demonstrated that the dynamic charcteristics of the soil deposits have induced an increase in the amplitude of the waves that arrive to the site where they have been recorded, in the region where the frecuency content of the record is close to the main excited frecuency of the soil deposit.

On the other hand, in cases where there has not been accelerogrphic records, like the 1942, 8.1 Richter magnitude Guayaquil, Ecuador earthquake, which epicenter was located in the Pacific Ocean at about 250 km NW from Guayaquil, on the subduction zone between the Nazca and South American plates, the effect of the soil appears clearly in the damage observed. Buildings with periods between 0.5 to 0.8 seconds located on the soft clay that underlines most of the city area, were heavily damaged and some even collapsed. Whereas those with smaller periods suffered perods suffered almost no damage (Ruffili, 1942)

On August 18, 1980 a 5.6 Richter Magnitude event, which epicenter was located on a secondary fault 65 km SE of Guayaquil (Lara, 1988), although it did not induce important damage to reinforced Concrete structures, it showed the same pattern of damage in downtown area as the 1942 event.

It is important to recognize that nowadays Guayaquil is a city with 150 km^2 and 2 million inhabitants whereas it was only 24 km^2 and 200.000 inhabitants in 1942, the year of the worst earthquake felt in the area, at least since 1860, therefore, the damage potential from severe earthquakes like that of 1942 is now quite greater then in the past

In order to have a better understanding that allows to predict the behavior of structures during earthquakes in the city of Guayaquil, ambient vibration tests were performed in 600 sites, most of them over the soft clay above mentioned and some over the rock that outcrops in a very reduced area of the city (Lara, 1996a). The statistical analysis of the aleatory records allowed the determination of the soil periods of the city.

Thanks to the small accelerographic network existing in Guayaquil, (Lara, 1987) there are some records from small to moderate earthquakes registered during the last 5 years and it is important to compare the frecuency content of the records with those obtained through the ambience vibration tests.

GEOTECHNICAL CHARACTERISTICS OF THE CITY

As expressed before, most of the city is on soft clay which depth varies from about 15 m to about 50 to 60 m. After the clay, there is a sand strata that increases its density with respect to depth.

The water table is located very high, at about 0.8 m from the surface, and this situation as well as the mechanical characteristics of the clay allows liquid limits between 85 and 130% as it was determined in most of the borings, being the upper limit the most frecuent value. The shear capacity of the soft clay is in the order of 0.01 to 0.02 N/mm^2.

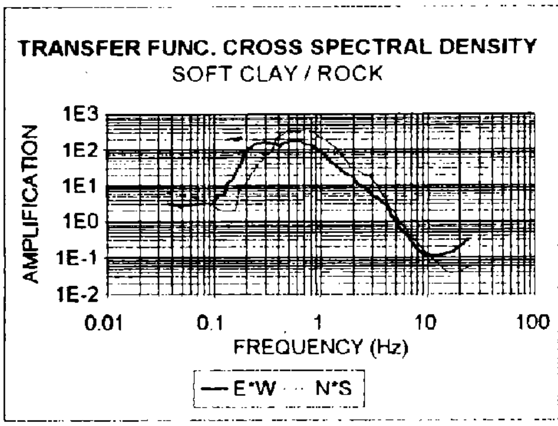
Due to the presence of the Chongon chain of mountains located at about 30 km west of the city, there are some rock outcrops in areas recently developed towards the NW part of the city. The shear resistance of this rock is about 0.2 to 0.3 N/mm^2.

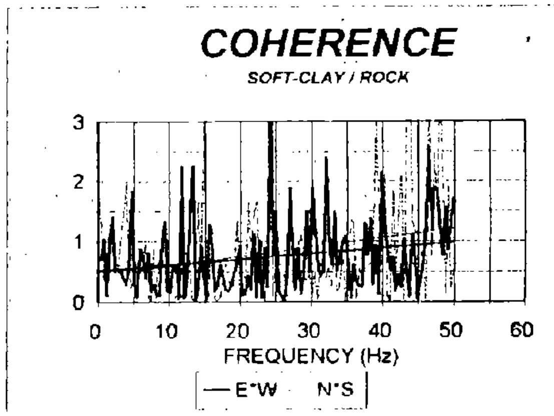
AMBIENT VIBRATIONS FRECUENCY CONTENT

Considering the recording at a rock site as the input signal or reference signal and the recording at any other site as the output, the transfer function between both cross-power spectral densities will be used to estimate site frecuency contents characteristics (Lara, 1996b)

Figure 1 shows the cross-spectral density transfer function between the soft clay and the rock sites for downtown area of the city and **figure 2** shows the coherence function for the same area. Notice that the average coherence varies from 0.5 to 0.6 for the range of frequencies under study (1.0 to 10 Hz), showing that the noise is moderate. **Figure 3** shows the isoperiods obtained through the cross spectral densities.

FIG. 1 AND 2 , RESULTS FROM AMBIENT - NOISE RECORDS





FRECUENCY CONTENT OF MODERATE EARTHQUAKE RECORDS

Considering the accelerograph records obtained after moderate earthquakes (Richter magnitudes between 4 and 5.1) which epicenters are located at more than 60 Km from Guayaquil, cross-spectral densities between soft clay and rock records were obtained. **Fig. 4** shows the transfer function of the cross-spectral density of a downtown record with respect to the rock site. Clearly the maximum energy content is in the same range of that showed in **figure 1** for ambient-noise record. Similar situations are observed in other transfer functions of cross-spectral densities related to accelerograms recorded in sites were ambient vibration records have been obtained.

Fig. 3 ISOPERIODS OF GUAYAQUIL CITY

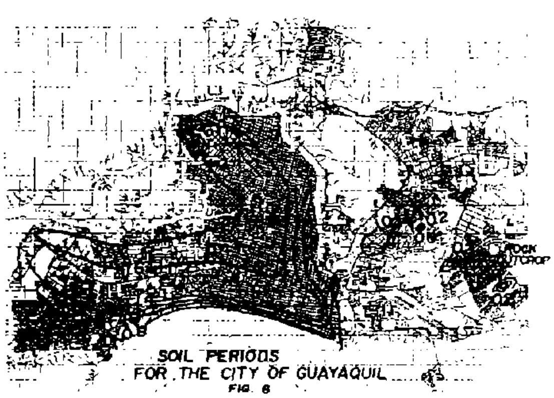
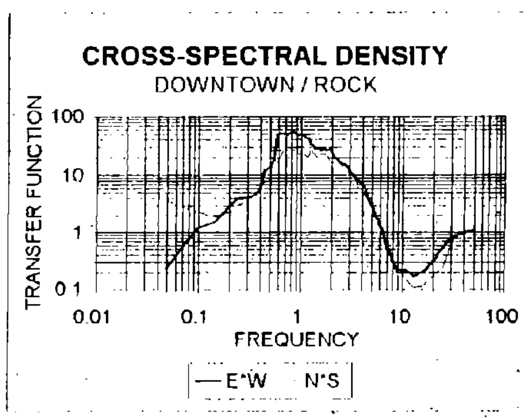


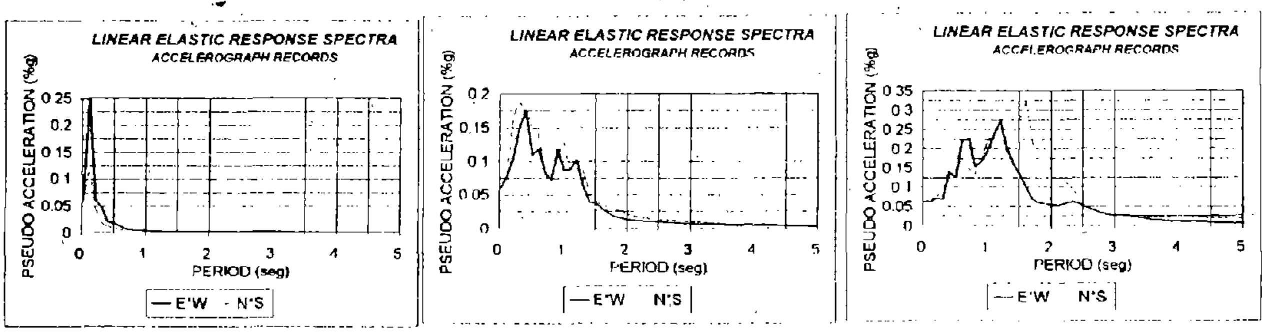
Fig. 4 TRANSFER FUNCTION
(Accelerograph Records)



LINEAR-ELASTIC RESPONSE SPECTRA

Based on these energy contents comparissons and using the moderate earthquake records above mentioned, normalized linear-elastic response spectras were obtained. **Fig. 5,6 and 7** show the linear elastic response spectras from moderate earthquake records, normalized to 0.06g for rock, intermidiate and soft clay sites which show, at 0.1g, amplifications corresponding to the following range of periods: 0.1 to 0.2 seconds, 0.2 to 1.2 seconds and 0.4 to 2.4 seconds respectevlcy.

FIGS. 5,6,7



CONCLUSIONS

The cross-spectral density procedure appears to be the best statistical parameter in order to obtain good estimates of soil periods and soil amplification factors.

Ambient-noise vibration tests provide very good results for estimation of soil dynamic characteristics when compared to moderate earthquake records.

Finally, the soil periods obtained through ambient vibration tests allow the establishment of the boundaries of linear elastic design response spectras for cities subjected to earthquakes and with different soil dynamic characteristics.

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Simplified Performance Based Seismic Design of Ductile Buildings Considering Damage Concentration

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SUMMARY

Tentative draft of simplified performance based seismic design of multistory ductile buildings with rigid diaphragm and parallel system including irregular-type mixed buildings is introduced considering acceptable inelastic deformation limits and several levels of major earthquake ground motions in Tokyo as an example.

INTRODUCTION

Simplified performance based seismic design methodology for multistory ductile buildings with irregularity is developed considering damage concentration in a particular story based on several current publications, technical papers and dynamic response analyses.

BASIS OF SIMPLIFIED PERFORMANCE BASED SEISMIC DESIGN

The shear and torsional strength capacity ratios Rx and Ry to the linear shear forces and linear torsional moments in the x- and y- directions, respectively, for each story of multistory ductile buildings with strong beam and weak column system considering damage concentration in a particular story are expressed as

$$Rx = \frac{A}{\dfrac{Q_{Lx}}{Q_{Px}} + \dfrac{T_{Lx} + BT_{Ly}}{T_P}} \quad , \qquad Ry = \frac{A}{\dfrac{Q_{Ly}}{Q_{Py}} + \dfrac{T_{Ly} + BT_{Lx}}{T_P}} \tag{1}$$

where Q_L and Q_P are the linear shear force acting on each story of the structure and the strength capacity of the corresponding story in the direction under consideration, respectively, when the structure is subjected to major earthquake ground motions.

The linear torsional moments acting on each story of the structure with respect to the center of strength S and the torsional strength of the corresponding story are denoted by T_L and T_P, respectively. Considering the orthogonal strength interaction of two directional seismic action, A=0.85~0.9 is used. Coefficient B is 0.3~0.5, considering the two horizontal components of major earthquake ground motions. In the above calculation, the effect of overturning moments can often be negligible to estimate inter-story drift ratios. However, two-steps calculation procedure is recommended especially for slender buildings using the strength capacity ratios and overturning moments obtained by the first step calculation, when each framing system of the structure is designed.

The following relationships can be obtained for the inelastic inter-story response of each framing system in the damage concentrated story to be within a certain acceptable deformation angle.

$$R_x > F_{Ds} \quad , \qquad R_y > F_{Ds} \tag{2}$$

where F_{Ds} is the linear response reduction factor in the direction under consideration for each framing system with various hysteresis loops after yielding and expressed as follows using non-linear response analysis:

$$F_{Ds} = [\,\theta_p / (\alpha_p\, \omega)\,]^m \qquad \text{or} \qquad \theta_p = \alpha_p\, \omega\, F_{Ds}^{\,1/m} \tag{3}$$

In the above equations, θ_P is elastic story drift ratio of each framing system at strength capacity level F_{QP}, and ω is acceptable inelastic deformation angle of the corresponding framing system of the story , respectively , as shown in **Figure 1**.

In **Figure 1**, the ratio of the strength capacity level F_{QP} to the linear lateral shear force F_{QL} is denoted by α_p. Coefficient m is obtained by hysteresis loop sizes after the yielding of structures with 3% viscous damping for steel structural systems and 5% for SRC and RC structural systems.

SEISMIC DESIGN OF MULTISTORY BUILDINGS CONSIDERING DAMAGE CONCENTRATION IN A PARTICULAR STORY

1. SEISMIC BASE SHEAR V

The seismic base shear V in a given direction is

$V = C_B\, W$

$C_B = S_1\, C_o$ mini.R_x

$C_B = S_1\, C_o$ mini.R_y

$C_B = S_1\, C_o$ For linear responses in the x- and y- directions.

where C_B: Design base shear coefficient.

W: The total weight of buildings.

S_1: Seismic coefficient for soil profile characteristics considering damage concentration in multistory buildings as shown in **Figure 2.**

C_o: Seismic coefficients as given in **Table 1.**

2. VERTICAL DISTRIBUTION OF SEISMIC FORCES F_x

The lateral seismic shear force F_X is determined in accordance with the following:

$F_x = C_{vx}\, V$

where

$$C_{vx} = \frac{W_x\, h_x^{\,k}}{\sum_{i=1}^{n} W_i\, h_i^{\,k}}$$

W_i and W_x are the portion of W located at or assigned to level i or x : h_i and h_x are the height above the base to level i or x: and k is an exponent related to the building period T. Modal analysis procedure is recommended for buildings with extremely vertical stiffness-irregularity.

3. HORIZONTAL SHEAR DISTRIBUTION

The seismic shear force at any level Vx is distributed to the various vertical components of the seismic resisting system in the story below level x with consideration given to the relative stiffness of the vertical components.

4. DRIFT DETERMINATION OF A STORY WITH mini.Rx and mini.Ry.

STORY DRIFT RATIO θp AT STRENGTH CAPACITY LEVEL FQP

	1	2	3	4	5	6
	SMRSF	EBSDF	MRSRCF	SRCDF	MRRCF	RCDF
ω	c/50	c/50	c/50	c/70	c/100	c/100
m	8.0	8.0	6.0	4.0	5.0	4.0
φ	1.0	1.0	1.0	0.8	1.0	0.8
θP						
FDs						
0.20	c/310	c/310	c/330	c/530	c/690	c/750
0.25						
0.30	c/200	c/200	c/210	c/320	c/430	c/450
0.35						
0.40	c/140	c/140	c/150	c/220	c/300	c/320
0.45						
0.50	c/110	c/110	c/120	c/170	c/230	c/240
0.55						

where c : Deformation control factor

φ: Capacity reduction factor as shown in **Figure 1.**

Minimum value of FDs is 0.25. Each framing system should be satisfied by the above story drift limitations depending on FDs.

SYMBOLS:

SMRSF(Special moment resisting steel framing system), EBSDF(Special moment resisting framing system with eccentric bracing system), MRSRCF(Moment resisting steel reinforced concrete composite framing system), SRCDF(Steel reinforced concrete composite dual framing system with infill shear walls), MRRCF(Moment resisting reinforced concrete framing system), RCDF(Reinforced concrete dual framing system with infill shear walls)

CONCLUSIONS

1. Seismic design procedure for multistory ductile buildings including irregular-type buildings with damage concentration subjected to certain levels of major earthquake ground motions is developed.

2. However, more research information is needed for the determination of seismic coefficients of soil characteristics, non-structural components, detailing of structures and so on.

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NEHRP RECOMMENDED PROVISIONS FOR SEISMIC REGULATIONS FOR NEW BUILDINGS, 1994 EDITIONS.

TABLE 1
SEISMIC COEFFICIENT Co
(TOKYO)

Co	Ground motions	Return period
0.50	AV=0.2 (25 kine)	50 years
0.75	Av=0.3 (35 kine)	100 years
1.00	Av=0.4 (50 kine)	200 years
1.40	Av=0.6 (70 kine)	500 years
1.70	Av=0.8 (90 kine)	800 years

FIGURE 1
FORCE-DEFORMATION RELATIONSHIP
(h : hight of story)

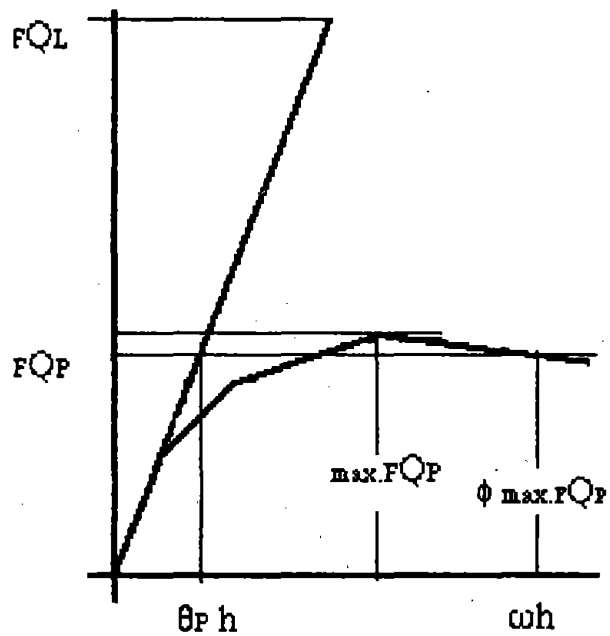
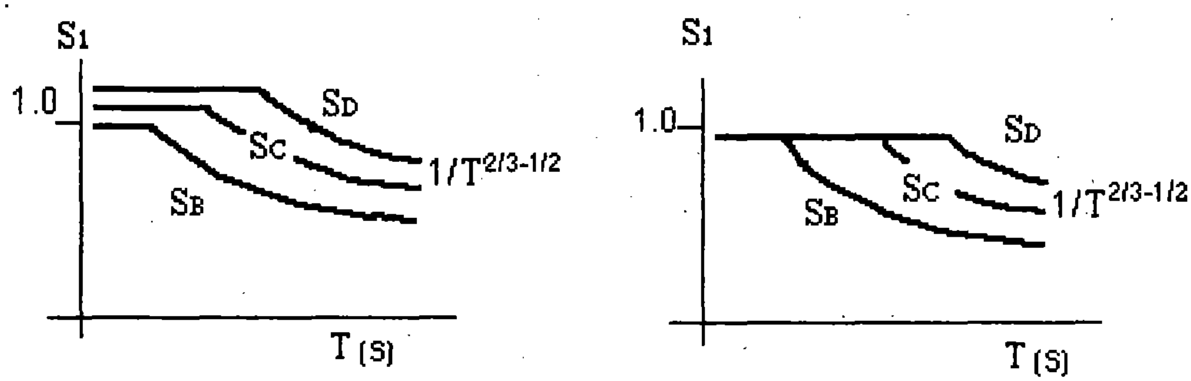


FIGURE 2
SEISMIC COEFFICIENT S1



Return period 50, 100, 200 years   Return period 500, 800 years
(SA, SE and SF are not specified.)

Architectural and Urban Configurations that Influence the Seismic Vulnerability of Buildings

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SUMMARY

Observation of buildings damaged by earthquakes has shown that architectural decisions related to aesthetics, function, cost, circulation, spatial relationships and other concerns affect the shape, dimension and location of structural and nonstructural elements, determine the existence and location of force-resisting components and cores, and influence other issues of significance in the design of earthquake-resistant buildings. Even urban planning decisions that refer to location, distribution and grouping of buildings within the structure of the city, overlook the special considerations that seismicaly hazardous areas require. Existing building codes are written for engineering application and rarely include guidelines related to architectural and urban features. Responding to urban planning principles and fad, urban zoning ordinances usually encourage and even enforce the use of certain configurations such as soft first stories, set-backs and adjacencies that, worldwide experience indicates, affect considerably the ability of buildings to withstand earthquakes. The inclusion of these features in the urban zoning ordinances of Caracas, Venezuela's capital, is an example of this situation that is a common practice in many modern metropolis located in seismic zones. This paper presents a brief description of these configurations that are encouraged and even enforced in the urban zoning ordinances of Caracas, and some recommendations for reducing the seismic vulnerability of buildings with such configurations, in order to mitigate seismic risk.

VULNERABLE ARCHITECTURAL AND URBAN CONFIGURATIONS

The detrimental effects of certain architectural design features, such as *soft first stories*, *set-backs* and *adjacencies*, in the response of buildings to ground motion has long been recognized by researchers in the design of earthquake-resistant buildings. Collective experience with building performance, based on observation of buildings that have been damaged in past earthquakes, has shown that these architectural and urban features can greatly influence the earthquake-resistant response of buildings. Damages identified in several buildings in the 1967 Caracas earthquake, are very illustrative examples of these conclusions. These configurations are usually included in the design of a building for the following reasons: (a) the most common, urban zoning requirements, originated by urban design limitations, (b) building functional program; and, (c) architectural and urban design and planning fad.

The following paragraphs illustrate a brief description of those configurations that are encouraged, and in some eases even imposed, in the urban ordinances of Caracas. More detailed descriptions are included in the documents mentioned in the References.

When the *soft first story* irregularity is present, two-situations arise: (a) one, it is created by a complete soft first story; for example, total absence of walls in the first story, because they were interrupted at the second story; this configuration is world widely used by architects, specially stimulate by Modern Architecture; (b) another, is the consequence of Post-Modern Movement, that incorporate internal setbacks in order to obtain double and even treble-high spaces in the lower floor; this architectural scheme derives, not only on a soft first story but on a diversity in the height of the first story columns, that many times impose the partial or total destruction of connectivities (beams and/or columns suppressed) at the lower stories. The soft first story scheme is typical in apartment buildings, hospitals and hotels. Some examples of dramatic collapse of buildings which present soft first story, are: (a) Olive View Hospital (San Fernando, California, 1971); (b) the Imperial County Services Building (El Centro, California, 1979); and, (c) several apartment buildings in Los Angeles (Northridge, California, 1994). The Caracas 1967 earthquake presents significant examples of structural damages in buildings with soft first story pattern, illustrated in the extensive bibliography related to this earthquake; the most dramatic, the Palace Corvin building.

In Caracas, apartment buildings are mostly designed with a moment resisting frame and the enclosures are mostly walls or hollowed masonry blocks. The lower story is usually left open to locate parking spaces and entrance hall; the upper stories accommodate the apartments. The zoning ordinances in force in Caracas stimulates the use of soft first story configuration. Those areas enclosed by soft first story are nor computable for habitability neither for total area limitations, neither for tax control, but they can be computable by the owner for selling purposes.

Soft first story is an architectural design scheme that is not to be eliminated from architects design criteria. It brings to the designer a series of functional and aesthetic advantages.

RECOMMENDATIONS

The following paragraphs present some brief recommendations that could be included in the ordinances in order to reduce the seismic vulnerability of buildings in which these configurations are unavoidable.

If *adjacencies* or continuous buildings are unavoidable in the urban distribution, then seismic joints are to be required. Building codes generally recommends that all structures shall be separated from adjoining structures. The size of the seismic joint can be calculated using the values given by code limitation on story displacement, or story drift. The joint might also be filled with a weak material which would easily crush so that any undesirable effect is avoided. The material which fills the seismic joint acts to damp the effects of buildings displacement and prevent the pounding between adjacent blocks. If the interstory heights, or the building heights of adjoining buildings, are different, special considerations must be taken in the design of the new structure and the reinforcement of the existing one, at the probable vulnerable points.

Setbacks have long been recognized as a problem and buildings codes in general, include special provisions for them. Zoning Ordinances should include guidelines in order to reduce the seismic vulnerability introduced by this irregularity Special care should be taken in order to avoid abrupt changes of strength and stiffness at the setback points and structural discontinuities in the lateral force-resisting system. The inverted setback configuration should be avoided in seismic areas.

Adjacency refers to the proximity of adjoining buildings, or adjoining parts of the same building such as blocks that are set back. When an earthquake occurs, these adjacent buildings, may move sideways at different periods due to their particular dynamic characteristics. This may result in one building or a building part "pounding" another, producing structural and/or non-structural damage.

An additional problem arises as a consequence of the vertical displacement of half levels between floors of *adjacent buildings* or building blocks. If sufficient separation is not provided, pounding of the floor slabs of one of the buildings into the story middlespan vertical supports of the other, can occur. It might also occur when adjoining buildings correspond to zonation ordinances that have changed on time, and the interstory heights, and even the building heights, are different. A tall building located between lower buildings in both sides, might produce an abrupt change of stiffness at roof level of the adjoining lower buildings, where lateral displacements of the taller buildings are constrained in its lower portions. and damage might occur at the point where a stress concentration occur.

This *adjacent building* configuration is frequently found in the downtown area of many cities located in seismic zones, because zoning ordinances enforce this urban solution type due to urban design requirements to provide volumetric continuity. Numerous examples of buildings damaged as result of this type of urban configuration were identified in Mexico City as consequence of the 1985 earthquake. Many of those buildings if they had not been adjoined to another building or buildings, would had performed accurately, or they would had not been damaged by the pounding of another.

Setbacks are, at present, one of the most common vertical irregularities in downtown areas. It consists of abrupt changes of floor size within the buildings height. A setback can be defined as a horizontal offset in the exterior vertical plane of a building. In many modern cities, zoning ordinance enforce this irregularity in the main commercial avenues and downtown, originated by urban design stipulations to provide adequate natural light and air to streets and neighboring buildings.

The seriousness of the *setback* effect depends on the dimension of the displaced parts of the building and the relative proportions. Setbacks are usually the cause of discontinuities in the lateral force-resisting system, due to the abrupt change of strength and stiffness that usually occurs at the point of the offset. Quite often when setbacks are present, columns are settled over the middle span of the lower floor beam. instead of maintaining the structural continuity. Setbacks containing shear walls that do not maintain the structural continuity to the foundation, may create severe problems. In addition, if the building is structurally asymmetric then torsional forces will he introduced into the structure, resulting in great complexity of analysis. Inverted setback buildings have additional problems since it usually create overturning moments.

The *soft first story* configuration is present in a building, when there is a significant change of strength and stiffness between the first lower floor and the upper floors. The earthquake forces are generally largest towards the ground floor, therefore, if a more flexible portion of the building is supporting a rigid and heavy mass, the majority of the energy will concentrate in the lower and more flexible story while the remainder small portion of the energy will be distributed among the rigid upper stories.

When the *soft first story* is unavoidable due to architectural programmatic limitations, the following recommendations taken from Guevara and Paparoni (1996) can be considered:

If it is a simple soft first story with total absence of walls in the first story, because they were interrupted at the second story; then; (1) use strong and stiff complete elevator and staircase cores, which can take all but the total base shear, leaving the first story columns almost only with axial loads; (2) use diagonals to stiffen the first story; (3) design the first story for much larger loads and smaller induced displacements than the rest of the structure, keeping the overall framed character of the building; (4) make "transitions" where the "softness" is distributed in several stories (this is very delicate task and needs careful tuning).

If the soft story is a consequence of Post-Modern Movement, related but not equal to the previous, and it arises when the architect wants to modify the facade frames only, leaving the inside ones as regular frames, be it with higher apparent story heights, be it by suppressing connectivities at the facade only, then: (1) the situation can be tolerated whenever the inner frames are complete, regular and sufficiently strong to dampen the local effects of the introduced framing irregularities (they must be at least on a 60% proportion of the total amount of framing); (2) in a totally framed structure, whenever the value of the following quotient is kept as constant as possible between successive floors, the effects of the irregularities will be minimized:

Total sum, of sectional Rigidities of all the columns/Total sum, of Floor Shear Rigidity

All the foregoing assertions can be considered as reliable under the condition of having very weak walls in the uppers stories, that is, that they will not increase the rigidity of the structural elements. When solid brick or rigid walls are used, most of these rules are not valid.

The large increase of the member forces in the first stories of buildings due to torsional effects tends to be ignored. Besides the dynamic influences, the simple fact that most of the first stories of buildings are designed as if they had built-in columns and theoretically rigid foundations, gives rise to very high concentrations of design forces there. In the case of seismic torsion, the additional effect of warping appears, due to the particular nature of most of the framing schemes in current use. When we add to that, sudden changes in rigidities caused by the disappearance of relatively rigid claddings over the soft story level, then large force concentrations appear, which can be attributed to torsional effects. As a conclusive remark, it is recommended to complete these facade frames with internal frames of sufficient strength and rigidity. The use of true space frames as structural schemes to determine member forces are also recommended.

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Seismic Retrofit of an Unusual Historical Arch Bridge Utilizing Coupled Shear Walls

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SUMMARY

Big Creek Bridge is a historical arch bridge near Big Sur California. This bridge has two fixed arch spans and two cantilever half arch spans located at either side of the fixed arch spans. The half arches terminating at the apex make Big Creek Bridge a unique structure in the world. The half arches are tied to each other with two sets of eye bar links buried in the two spandrel girders. This bridge is completely self balanced under its own weight (i.e. the horizontal thrusts of the arches are resisted by the eye bar chains at the deck level.) Self balancing design was developed to avoid relying on the soil resistance at the abutments. Considering the highly fractured Franciscan Melange rock formation at the abutments and the non-existence of geotechnical engineering theory at its design date (1938) the Big Creek Bridge was far ahead of its time.

The preliminary diagnostic seismic analyses indicated several vulnerabilities of this bridge including the inadequate deformation capacity of almost all members and failure of the eye bar chains possibly leading to overall collapse.

As the first step of the retrofit the articulated deck was converted to a continuous diaphragm by post tensioning. At the abutments four large diameter CIDH Piles were designed to transmit the deck forces to the foundations.

Afterwards, transverse coupled shear walls were designed to limit the lateral displacements and thus the deformation requirements on the existing spandrel bents and arch rib struts. Shear walls with link beams (coupled shear walls) provided the right stiffness for the structure and the added energy dissipation. The high tensile chord forces lead to post tensioning of coupled walls and addition of tiedowns at the foundations.

To calculate the demands on the retrofitted structure spectral analyses were carried out. The secant stiffness of yielding substructures such as the coupled walls were calculated from inelastic pushover analyses.

By utilizing coupled walls it was possible to limit the deformations at the bases of the existing piers (lap splice zones) and reduce the forces transmitted to the foundations.

THE STRUCTURE

Big Creek Bridge is a 587 foot-long historical arch bridge near Big Sur, California which carries two lanes of traffic on State Highway 1. The structural configuration is unique as Big Creek Bridge is the only arch bridge in the world that has two (cantilevered) half-arches terminating at the apex. Structurally the bridge has two 160 foot fixed arch spans, two 85 foot cantilevered half arches terminating at the apex and two articulated drop-in end spans, (see **Fig. 1**). The cantilever half arches have hinge castings buried at the skewbacks and eye-bar-chains running at the apex (deck level), tying the two ends of the half arches together, (see **Details A, B and C in Fig.1**).

In between the terminal points of the arch ribs (skewbacks) there are three 100 foot-tall Piers consisting of two shear walls 18 feet apart from each other in the transverse direction. The arch ribs and pier bases merge together to share a common spread footing. The uniqueness of this bridge stems from the fact that the horizontal push of the arches is balanced by the eye-bar chain connecting the apexes of the half arches (at the deck level). The shear walls have partial height vertical expansion joints separating the end columns from the webs. Spandrel columns with longitudinal concrete hinges and partial height vertical joints in the shear walls were utilized to reduce the thermal stresses on the arch ribs and on the rest of the structure. The only elements holding the cantilever end spans in their places and transferring the live loads are two steel eye bars running between the apexes of the two half arches, tying them together. The relative flexibility of the structure in the longitudinal direction, inducing very little thermal restraint, causes the structure to be too flexible under dynamic seismic actions. In the transverse direction there are spandrel column struts, arch rib struts and pier wall struts of varying dimensions. These struts are of typical 1930's design utilizing square reinforcing bars, big haunches at the ends and very little transverse steel.

To protect the bridge from slope failures near the abutments the articulated end spans were, originally, designed to fall off from their seats leaving the rest of the bridge intact.

In addition to its structural ingenuity and uniqueness Big Creek Bridge teaches us a lesson in bridge aesthetics by being very transparent. The spandrel columns, although placed at every 16 feet, do not impede the pristine ocean view.

THE RETROFIT

THE DESIGN CRITERIA AND A POSSIBLE DAMAGE SCENARIO

Caltrans Arch Committee (1995) recommended the following design criteria which was more of a performance criteria:

- "Retrofitted structure shall not collapse after a major seismic event" This major seismic event was defined by using Caltrans Acceleration Response Spectrum Type A for 0.6g, as required by (Caltrans BDS), (Caltrans, MTD), (Caltrans BDA), (Caltrans BDD).
- "The retrofits to arch ribs should be avoided as much as possible"
- "Displacement based design method of Priestley (1995) may be utilized for economy"

Dynamic spectral analyses of the structure in "As-Built" condition confirmed that longitudinal hinges in the spandrel columns make the structure too flexible longitudinally causing the deck to deflect up to 12", resulting in failures of the end spans and of the "tie bars" holding the half arches together. In the transverse direction failures were predicted in almost all of the transverse struts and pier bases because of inadequate deformation capacities. To summarize: this bridge would likely to collapse if an earthquake similar to the design earthquake were to occur.

THE RETROFIT STRATEGY

A very common problem in pre-1945 R/C designs is the presence of large size (1 1/8" or 1 1/4") square Intermediate Grade reinforcing bars. These Grade 40 bars do not have the high ridges of modern deformed reinforcing steel. There are very few informative resources regarding the strength characteristics of R/C sections reinforced with square bars. CRSI Publication on reinforcement in historical concrete structures (CRSI 1980) suggests that the development lengths of square bars be assumed to be twice the development lengths of modern bars with comparable areas. Therefore it was concluded that the 35 diameter lap splices shown on the as-built structural plans cannot develop the deformation requirements from 1 1/4" square bars as required by a ductile design. Confinement of plastic deformation zones, although prohibitively expensive, is one of the remedies and, limiting maximum tensile and compressive strains in the sections is the other alternative.

To improve the seismic response of the Big Creek Bridge two retrofit schemes were analyzed and the resulting damage in the structure was evaluated with respect to the cost of the retrofit and the predictability of the behavior of the retrofitted structure.

- The first scheme consisted of constructing large diameter CIDH abutments, providing deck continuity with post tensioned edge beams and confinement of pier, arch rib and some of the spandrel bent struts and columns.
- The second scheme consisted of constructing large diameter CIDH abutments, providing deck continuity with post tensioned edge beams and constructing transverse shear walls at the pier locations.

The preliminary design strategy utilizing the transverse shear walls (scheme number 2) was economically competitive, and, as compared to the first alternative, produced a structure that had a more predictable behavior.

THE DETAILED DESIGN

CONTINUOUS DECK AND CIDH PILE ABUTMENTS

The deck in its as-built condition, has 9 separate pieces connected by expansion joints. In order to make the deck act like a transverse diaphragm, two continuous post tensioned edge girders were designed that would run parallel to the existing (but discontinuous) spandrel girders. These edge girders were then connected to 4 CIDH Piles at each end via an abutment slab. Therefore a positive load path was provided both in the longitudinal and transverse direction that would transmit the deck loads to the new large diameter CIDH Pile Abutments, (see **Fig. 2**).

Although ensuring the deck continuity by using post tensioning is a simple concept, the variable spandrel girder depth as well as slight horizontal curve of the bridge caused the deck to be evaluated for all possible serviceability as well as ultimate level load combinations, including the thermal effects.

SHEAR WALLS: HOW MUCH STRENGTH IS TOO MUCH?

The use of transverse shear walls parallel to post tensioned (continuous) deck would limit the transverse displacements and thus reduce the damage in the spandrel bents and at Pier Bases. During the course of detailed design it was noticed that infilling the piers with transverse shear walls made the structure attract the maximum possible seismic forces. If we were to use infilled shear walls the foundations would not be able to carry the attracted forces without extensive retrofit and abutments could not be designed. The stiffness (or slenderness) of abutments are critical as stiffer abutments would have caused larger thermal force variations in the deck.

Because the bridge spans an environmentally sensitive area it was not possible to enlarge the existing footing dimensions. Therefore the forces induced by the shear walls on the foundations were limited to the maximum bearing capacity of the foundations. The strength of the shear walls were then calculated based on the foundation capacities. Although the coupled walls significantly softened the seismic response of the structure, the forces were still too high to be designed by utilizing ordinary Reinforced Concrete. Therefore, post tensioning tendons in the coupled shear walls, as well as large tiedowns in the foundations, were utilized to transfer the large chord forces into the foundation medium, (see **Fig. 3**).

Large openings were designed within the transverse shear walls resulting in a coupled-wall system. By utilizing coupled walls :

- It was still possible to control the displacements effectively,
- It was still possible to control the tensile strains at the bases of the piers,
- It was possible to dissipate energy via plastic deformation of the link beams.

As opposed to infilling piers with transverse shear walls completely, the openings between the coupling girders and shear walls provided some transparency for the coupled shear wall design.

ANALYSIS ISSUES

Linear dynamic analyses were carried out to capture the global behavior of the structure. The secant stiffnesses of important members were calculated by using 2D inelastic push analyses (Drain2DX) and the appropriate stiffnesses were then incorporated into the global dynamic model. Since the global analyses are linear a few iterations are necessary to converge to the right displacement.

At the as-built vulnerability phase the global dynamic model was a diagnosis model and attempted to model every component of this complex bridge. However as the design progressed, the global dynamic model was simplified significantly and was geared towards providing design displacements for the critical components. For example in the global dynamic analyses the coupled walls were modeled by using a single column element which is actually, a superelement. The inelastic behavior of the coupled walls were modeled by tuning the stiffness of this single column super-element to the stiffness of the coupled wall substructure as obtained from the nonlinear pushover analyses. The increased damping due to inelastic action of the link beams was not modeled.

CLOSURE

As the design progressed the need for the simplest possible dynamic model became more pronounced. Simple super-elements that were tuned to the appropriate stiffnessess of nonlinear substructures (such as coupled shear walls) were adequate. It was observed that any changes to the load carrying system such as closure of expansion hinges, introduction of new shear walls etc., should be carefully evaluated and all of the applicable load cases (including serviceability) should be checked.

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GLOSSARY

CIDH:	Cast In Drilled Hole
Skewback	The surface perpendicular to the centerline of the arch rib at the face of the foundation.
Spandrel Bents	Bents supported by arch ribs.

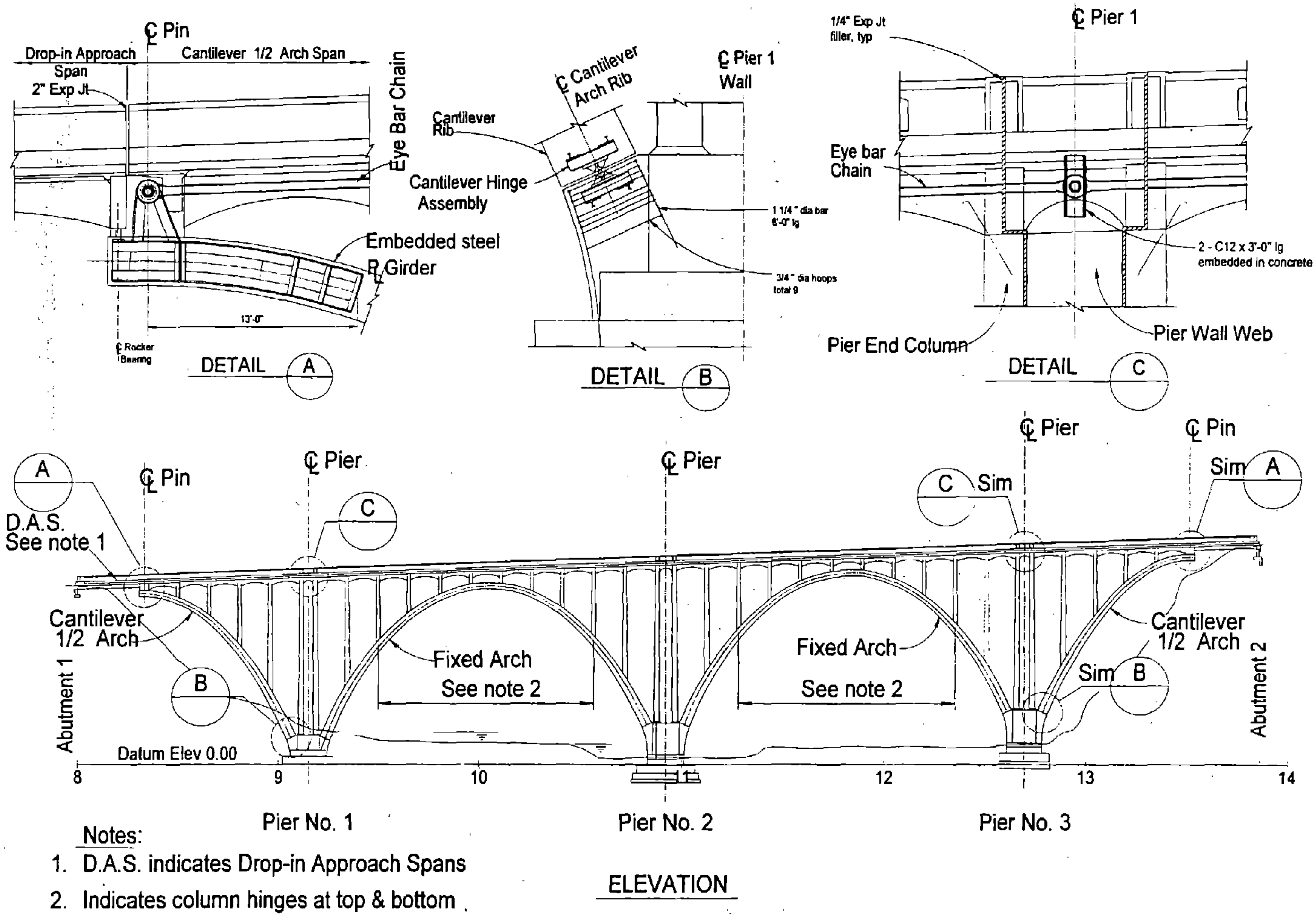


FIG. 1: BIG CREEK BRIDGE: DESCRIPTION OF THE STRUCTURAL SYSTEM

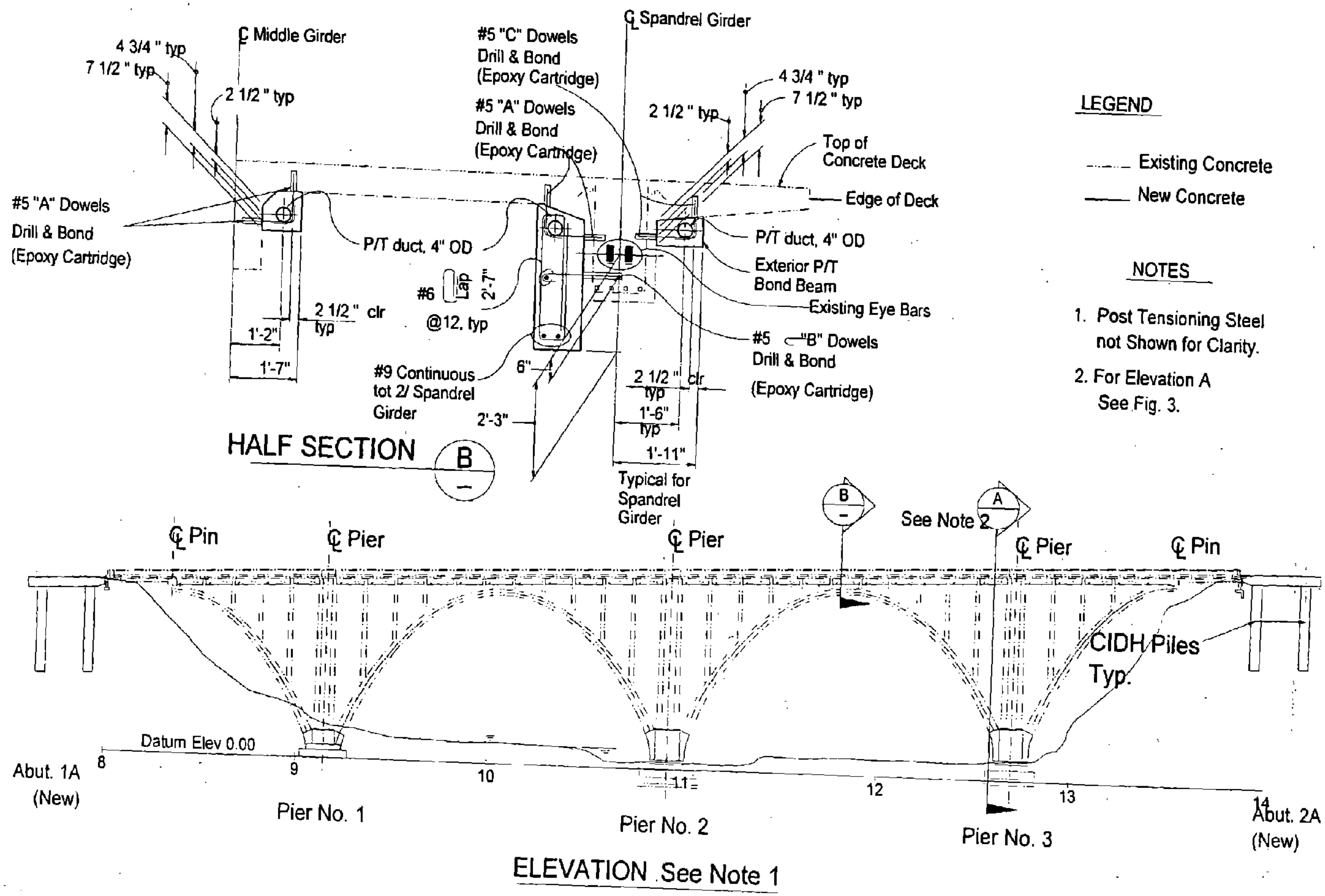
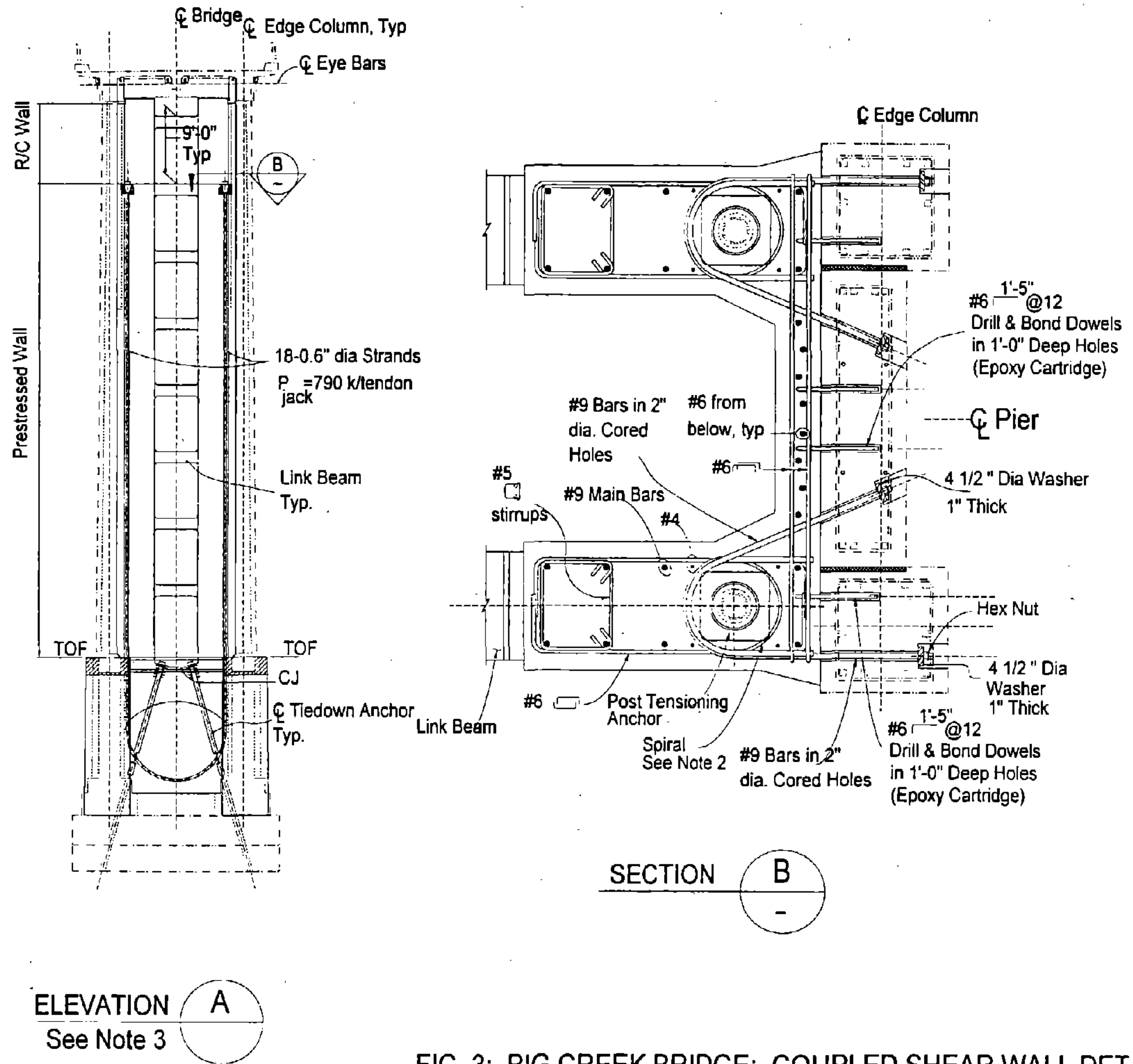


FIG. 2: BIG CREEK BRIDGE: TYPICAL DECK RETROFIT



LEGEND

- - - - Existing Concrete

——— New Concrete

NOTES

1. Post Tensioning Steel Shown with Solid Lines For Clarity.
2. In Elevation A Shear Wall Reinforcement Not Shown for Clarity.
3. Elevation A Defined in Fig. 2.

FIG. 3: BIG CREEK BRIDGE: COUPLED SHEAR WALL DETAILS

How to Quantify Performance Criteria — A Discussion

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SUMMARY

Many papers have been published in recent years on performance-based seismic design criteria and the need to codify these concepts. One of the first attempts to codify performance-based design appears in Vision 2000 where building performance levels are quantified in terms of component damage for different building types. Performance levels for concrete construction are characterized by the distribution and width of cracks in various components. Implicit in this approach is the idea that component damage can be directly correlated with global building behavior and therefore can be used to quantify performance levels. This paper presents a discussion of an alternative method to quantify structural performance; one that considers global building performance explicitly. Building systems are characterized as having ductile, semi-ductile, or brittle behavior. Structural performance levels for each system are defined in terms of global displacement ductilities obtained from either a nonlinear or push-over analysis that considers the inelastic behavior of the structural system. The paper concludes with suggestions for additional research required in order to correlate performance levels with global displacement ductilities for the three basic types of building systems.

BEHAVIOR CATEGORIES FOR BUILDING SYSTEMS

Many lateral force resisting systems developed in recent years use combinations of materials such as steel chevron braces with visco-elastic dampers, structural steel shapes embedded in concrete frames or shear walls, or concrete columns combined with steel beams. Thus, it appears more useful to categorize lateral force resisting systems by their expected behavior or ductility rather than by material type. The range of expected behavior can be divided into three categories--ductile, semi-ductile or semi-brittle, and brittle behavior. Using this scheme, the structural behavior category may depend on several factors including materials of construction, structural configuration, quality of detailing, and compatibility of architectural and nonstructural elements with the structural system.

FORCE-DEFLECTION RELATIONSHIPS

Force-deflection curves provide a clear illustration of the differences between ductile, semi-ductile, and brittle behavior. **Figure 1** presents a generalized force-deflection curve for a structure with ductile behavior. The figure shows base shear vs. roof displacement for the full range of behavior from initial loading to collapse and identifies four critical points that define transitions between different ranges of behavior. These critical points occur at first yield, at major yield, at initial deterioration, and at collapse. These points can be used to divide the force-deflection curve into four ranges of structural behavior as follows: 1) service range, 2) yield range, 3) damage range, and 4) survival range. The critical points and ranges of behavior are illustrated in **Figure 1**. The first two ranges, from initial loading up until the onset of major yielding, are often referred to as the elastic range. Beyond this point, the structure exhibits inelastic behavior in the damage and survival ranges. Either nonlinear or push-over analyses are required to generate the data needed for the force-deflection plot shown.

CAPACITY SPECTRUM METHOD

In order to obtain a clear picture of a building's performance during an earthquake, it is useful to define both the seismic demand and the performance as a function of inelastic behavior. The Capacity Spectrum Method described in Tri-Services (1996, p. 7-1) is a graphic method that compares the earthquake demand with the global capacity of the structure. The demand is the site response spectrum expressed in terms of spectral acceleration, S_a, and spectral displacement, S_d. The building capacity is given by a force-deflection curve similar to **Figure 1**. To make the comparison simpler, the acceleration is converted into a base shear force using the modal masses or vice-versa. The point where these two curves cross defines the level of inelastic behavior reached when the building is subjected to the given earthquake as illustrated in **Figure 2**.

PERFORMANCE-BASED SEISMIC DESIGN CRITERIA

The seismic design criteria contained in U.S. building codes have been based on objectives stated in SEAOC (1996, p. 93) that buildings designed in accordance with minimum code standards should resist minor earthquakes without damage, moderate earthquakes without structural damage but with nonstructural damage, and major earthquakes with possibly both structural and nonstructural damage but without collapse. The link between these qualitative objectives and the design requirements in the code has been based on observed damage from a number of historic California earthquakes and a fair bit of wishful thinking. It is now widely acknowledged that a more rigorous methodology is needed, one based on quantitative relationships between a particular design approach — including design forces, detailing, configuration, redundancy, compatibility — and actual performance.

The desired performance of a building depends on the building function and the degree of damage and functional loss that building owners, insurers, and tenants are willing to accept following an earthquake. Buildings are categorized by an occupancy type or function as Essential, Hazardous, Special Occupancy, or Standard Occupancy facilities. These categories are used to assign an Importance Factor, I, that in turn is used to compute the base shear force for seismic design. Currently, the I factor is 1.25 for Essential and Hazardous facilities, otherwise 1.0, and the code does not contain any other mechanism to provide a lower level of damage or higher level of performance.

The following discussion is an attempt to define the seismic performance of building systems on a more rational basis, one which correlates desired building performance with the extent of inelastic behavior required to survive a particular earthquake. For each of the four occupancy categories cited above, the desired performance is given for Major, Moderate, and Minor earthquakes. Essential facilities: a) Major — very little damage, most of the structure undamaged, displacements kept to a level that do not affect operations; b) Moderate — no damage; c) Minor — no damage. Hazardous facilities: a) Major — structure may suffer light to moderate damage, displacements kept to a level that do not cause a shutdown or affect operations; B) Moderate — no damage; c) Minor — no damage. Special Occupancy: a) Major — structure may suffer moderate damage, displacements and structural distress should be kept to a level that allow occupants to exit safely without panic; b) Moderate — structure may suffer light to moderate damage, damage repairable; c) Minor — light to no damage. Standard Occupancy: a) Major — structure may suffer severe damage, facility may not be usable or repairable; b) Moderate — structure may suffer moderate damage but facility usable and damage repairable; c) Minor — light damage, minor cosmetic repairs may be required.

PERFORMANCE EXPRESSED IN TERMS OF GLOBAL DISPLACEMENT DUCTILITY

The degree of inelastic behavior can be expressed in terms of a Global Building Displacement Ductility (GBDD). The GBDD is defined as the roof displacement during the earthquake divided by the roof displacement at yield. Based on the authors preliminary review of available test data, **Figures 3, 4, and 5** are a first attempt to quantify the levels of GBDD that will give the desired performance described above for ductile, semi-ductile, and brittle structures, respectively. The service, yield, damage, and survival ranges are indicated in each figure along with occupancy categories. These figures indicate that the GBDD at the beginning of the survival range of performance is on the order of 4 for a ductile structure, 2.5 for a semi-ductile structure, and 0.8 for a brittle structure. These are estimates for a major earthquake and would need to be reduced for moderate and minor earthquakes.

In order to implement the proposed scheme for performance-based design, additional research is needed to quantify appropriate ductility levels for different types of structures. This would include a request to major testing facilities to clearly document the deterioration of test specimens from first yield to collapse with pictures and detailed damage descriptions at distinct ductility levels and include this data in published reports. A more thorough review of existing reports is needed in order to prepare a data base of behavior vs. ductility from experimental data. An evaluation of earthquake damage data from instrumented buildings is needed to correlate behavior vs. ductility. These efforts would help to establish quantitative relationships between ductility and performance and provide an improved basis for performance-based design criteria.

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FIGURE 1 FORCE -DEFLECTION CURVE

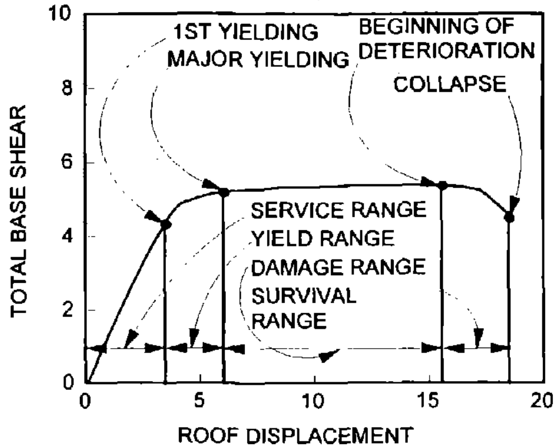


FIGURE 2 CAPACITY SPECTRUM METHOD

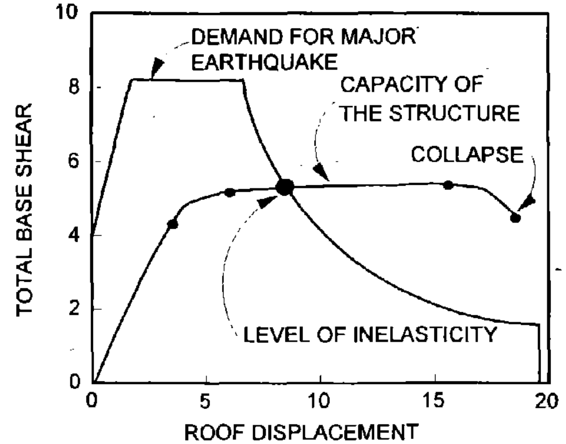


FIGURE 3 DUCTILE BEHAVIOR

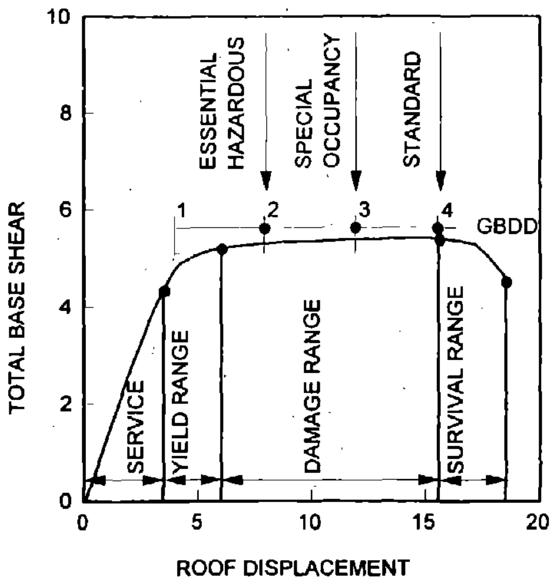


FIGURE 4 SEMI-DUCTILE BEHAVIOR

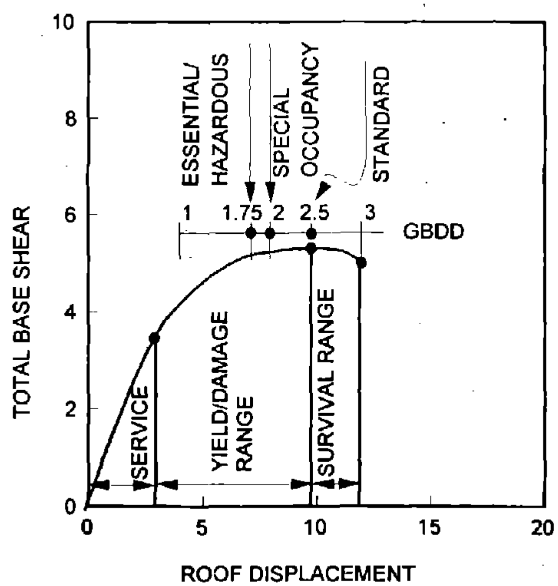
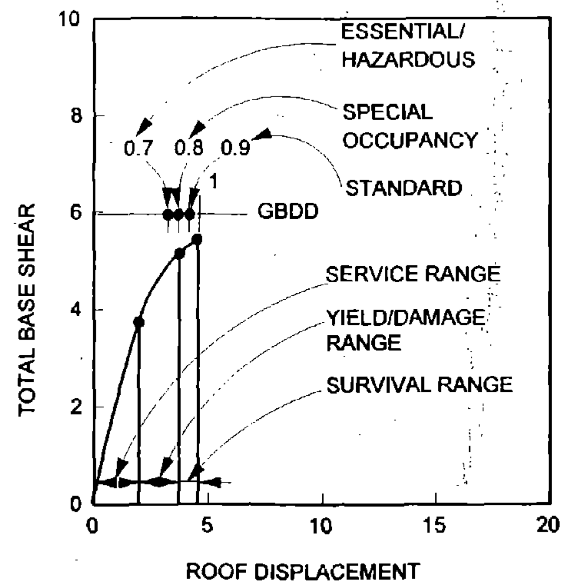


FIGURE 5 BRITTLE BEHAVIOR



Steel/Composite Coupling Beams

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SUMMARY

Results from an ongoing research on seismic performance of steel-concrete composite coupling beams are presented. This study suggests that current guidelines are overly conservative by ignoring the contribution of concrete encasement towards stability of web and flange at advanced yielding, but unconservative for design of beam-wall connection and for establishing beam stiffness. The contribution of floor slab to stiffness diminishes at low deformations and can be ignored. However, the slab improves the performance of beam-wall connection. As expected the compressive area of floor slab increases the flexural capacity of composite coupling beams, but the contribution of tensile force in slab bars is not significant. Design recommendations are made.

INTRODUCTION

Extensive past research (e.g., Aktan and Bertero, 1981) has led to well established guidelines for seismic resistant design of reinforced concrete coupling beams (NEHRP, 1994). Due to their simpler details, steel or steel-concrete composite coupling beams provide a viable alternative. The steel beam is typically embedded into the wall, and is encased over its span between the wall piers. Using models developed originally for steel brackets in precast columns (e.g., Mattock and Gaffar, 1982), the required embedment length is computed to develop beam shear capacity. According to current guidelines (NEHRP, 1994), steel coupling beams are designed as shear links although the detailing is based on an assumed shear angle of 0.09 rad. At this angle, closely spaced web stiffener plates are required for cases with low span to depth ratios. Note that for shear links the expected level of shear angle is computed, and the member is detailed accordingly. The effects of floor slab and concrete encasement around steel coupling beams are also ignored.

A research program at the University of Cincinnati has been examining issues related to cyclic behavior of composite coupling beams in an effort to improve available design guidelines. The effects of concrete encasement and floor slab were among the main parameters in this study. This research is summarized herein, and important observations are presented.

SUMMARY OF RESEARCH PROGRAM

The research revolved around a 20-story prototype structure. The primary lateral resisting system consisted of three reinforced concrete walls that were coupled to form a central core. A total of seven subassemblies were selected and tested. The main test variables were (a) presence or lack of concrete encasement which was nominally reinforced, (b) number and spacing of web stiffener plates, (c) the level of coupling beam force for which the embedment length is computed, (d) presence or lack of floor slab, and (e) the level of wall overturning moment.

TEST RESULTS AND DISCUSSION

The test results are described with reference to four specimens which are summarized in **Table 1**. Face bearing plates in specimens No. 6 and 7 were a pair of stiffener plates (on both sides of the web) placed along the embedment length to mobilize compression strut in the connection.

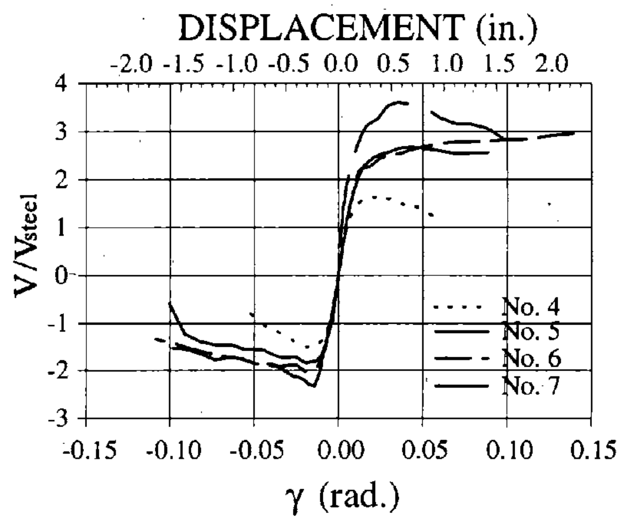
TABLE 1
TEST PARAMETERS

Specimen I.D.	Concrete Encasement	Web Stiffeners	Face Bearing Plates (FBP)	Consider Encasement for Computing L_e	Floor Slab
4	Yes	No	No	No	No
5	Yes	No	No	Yes	No
6	Yes	No	Yes	Yes	No
7	Yes	No	Yes	Yes	Yes

L_e = Embedment length of coupling beam inside wall

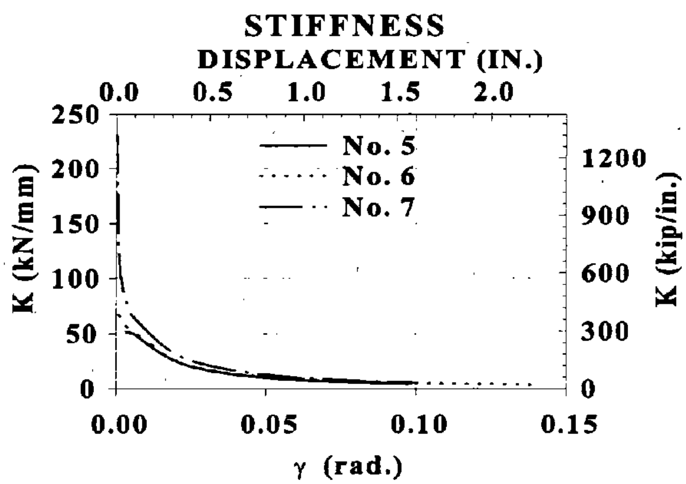
The load-deflection envelope curves are compared in **Figure 1**. In this figure, V_{steel} is the plastic shear capacity of the steel coupling beam. All the specimens could develop and exceed V_{steel} even though web stiffener plates were not used. The lightly reinforced encasement around the steel beam was adequate to prevent web buckling at advanced yielding. Current design guidelines appear to be overly conservative by ignoring the beneficial effects of concrete encasement. Nevertheless, the mode of failure in specimen No. 4 was due to excessive damage in the connection region (Gong et al., 1996), which is less desirable than dissipating the input energy through plastic hinges in the coupling beam. This failure is attributed to the fact that the embedment length was calculated to develop the beam shear capacity without any regard to the concrete encasement. Following capacity design concepts, one would realize that the embedment length needs to be adequate to delay connection failure before fully mobilizing the coupling beam's contribution towards energy dissipation. Based on a number of parametric studies, it is suggested that the required embedment length be established to develop 1.75 $(V_p + V_c + V_s)$ where V_p = plastic shear capacity of steel beam, V_c = shear capacity of concrete encasement, and V_s = shear capacity of web reinforcement. Since material over-strength factors have been incorporated in the proposed equation, V_p, V_c, and V_s are computed based on nominal material properties. As seen from **Figure 1**, the longer embedment length (computed based on the proposed method) enabled specimens 5 and 6 achieve larger loads at higher shear angles, and experienced significantly less strength reduction. The presence of face bearing plates improved the ductility but not the capacity. As expected, the compressive cross-sectional area of the floor slab led to a higher load for specimen No. 7. However, when subjected to tensile stresses the slab participation was insignificant as the additional tensile forces in the slab bars are small in comparison to the tensile force in the steel coupling beam flange. Although slab participation was ignored when computing the embedment length, the overall response of specimen No. 7 was similar to that for specimens No. 5 and 6. The floor slab reduced the participation of the connection region, and hence the provided embedment length was sufficient.

FIGURE 1
COUPLING BEAM SHEAR-SHEAR ANGLE ENVELOPE CURVES



As noted elsewhere (Gong et al., 1996), the encasement around steel coupling beams can appreciably increase the initial stiffness. The increased stiffness needs to be incorporated to avoid un-intentional over-coupling of walls. **Figure 2** indicates that the floor slab increases the initial stiffness by a factor of 3.5. However, at small deformations (beam shear angle = 0.005 rad.) the stiffness drops to a level comparable to that for specimens No. 5 and 6. This observation suggests that the effects of slab can be ignored in so far as the stiffness of composite coupling beams is concerned.

FIGURE 2
VARIATION OF COUPLING BEAM STIFFNESS

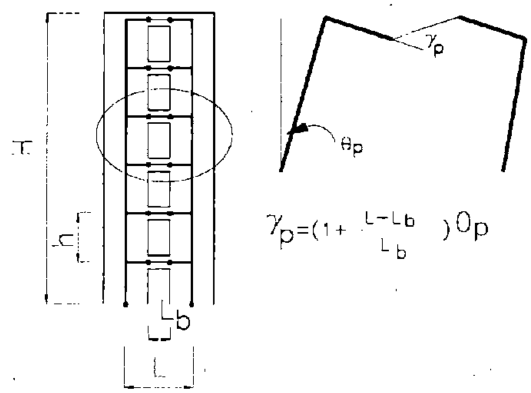


EVALUATION OF SHEAR ANGLE

Current guidelines (NEHRP, 1994) arbitrarily set coupling beam shear angle to 0.09 rad., and the beam is detailed accordingly. Detailed dynamic and static analyses indicate that for reasonably proportioned coupled walls, the maximum coupling beam shear angle is significantly smaller than this level. In an attempt to compute the expected shear angle more rationally, the assumed collapse mechanism shown in **Figure 3** is proposed. The value of θ_p (plastic interstory drift angle) is taken as $0.4R\theta_e$ where θ_e = elastic interstory drift angle computed under code level lateral loads and R = code specified response modification coefficient. Knowing the value of θ_p, shear angle (γ_p) is θ_p L/L_b. With the exception of the assumed collapse mechanism and the relation between shear angle and drift angle, this method is similar to that used for steel shear links.

Note that L_b (length of coupling beam) should account for the flexibility of steel coupling beam inside the wall, and is not identical to the clear span between wall piers (Shahrooz et al., 1989).

FIGURE 3
MODEL FOR COMPUTING SHEAR ANGLE



SUMMARY AND CONCLUSIONS

Several shortcomings of current design guidelines for steel-concrete composite coupling beams were noted. Detailing can be simplified if the beneficial effects of concrete encasement around steel beams are taken into account, and the method proposed in this paper is used to compute the expected beam shear angle. The steel beam must be embedded adequately in the wall so that the capacity of the composite beam is developed and not just the capacity of the steel beam. However, the additional strength due to floor slab may be ignored since it is relatively small, and the slab helps to form plastic hinges in the beam rather than in the connection region. The contribution of slab towards composite coupling beam stiffness is lost after the beam undergoes small deformations, and may be neglected..

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Acceptability Checks For Performanced Based Design

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SUMMARY

Since Performance Based Design has been introduced some concern has arise in the profession regarding the possible large number of analysis required to verify if the design satisfy the performance objectives, as well as how to evaluate the design reliability or the target failure probability associated to each performance objective. In this paper a rational approach to manage these problems as well as a practical methodology for doing acceptability checks for linear and non-linear analysis are presented.

INTRODUCTION

The first step of the comprehensive design approach is the selection of the performance objectives. These are selected and expressed in terms of expected levels of damage resulting from expected levels of EQGMs. This selection is made by the client in consultation with the design professional based on consideration of the client's expectations, the seismic hazard exposure, economical analysis and acceptable risk. A design performance objective couples expected performance level with levels of possible seismic hazard, as illustrated in the Performance Objective Matrix (SEAOC 1995). Performance levels are defined in terms of damage to the structure and non-structural components, and in terms of consequences to the occupants and functions of the facility. The performance levels can be as follows : *Fully Operational or Serviceable* (facility continues in operation with negligible damage) ; *Operational or Functional* (facility continues in operation with minor damage and minor disruption in non-essential services) ; *Life Safety* (life safety is substantially protected, damage is moderate to extensive) ; and *Near Collapse or Impending Collapse* (life safety is at risk, damage is severe, and structural collapse is prevented). The seismic hazard at a given site is represented as a set of EQGMs and associated hazards with specified probabilities of occurrence *(frequent, occasional, rare and very rare).*

Performance objectives typically include multiple goals. For example, for a particular building and site they may be those indicated in **Table 1. Acceptability checks to verify that the selected performance objectives are met are central to the performance based design strategy** (Bertero et al. 1996). In a particular building, the design of specific components may be controlled by the same or different response parameters for the same or different performance objectives. Typical response parameters may include : stress ratios, deformations, rate of deformations (velocity, acceleration and jerk), interstory drift ratios, ductility demands ratios, damage index, and energy dissipation demand vs. capacity.

TABLE 1
PERFORMANCE OBJECTIVES

	EQGM's Level Return Period TR (years)	*Struct. Damage Max Local DM index*	*Failure probability P_f*	*Non-str. damage Max IDI Θ*	*Failure probability P_f*
Serviceable	30 years	0.2	0.20	0.003	0.30
Operational	75 years	0.4	0.20	0.006	0.30
Life Safety	475 years	0.6	0.10	0.010	0.20
Near Collapse	970 years	0.8	0.10	0.015	0.20

Typical limiting values for these response parameters must be established for each performance level through research, including laboratory testing of specific components and calibrating the limiting values by analyzing buildings whose EQGMs and responses, and therefore damages, have been measured in past EQs. Then, in a specific design, the appropriate parameters must be checked at the governing performance levels. Acceptability checks under the EQGMs corresponding to the service performance level are straightforward because as the building should remain essentially in its linear elastic range, the principle of superposition applies. On the other hand, for the other three performance design objective considered, non-linear analysis should be done for each level of EQGMs. Therefore, some concern exists in the profession regarding the possible large number of analysis required to verify if the design satisfy the performance objectives, as well as how to evaluate the design reliability or the target failure probability associated to each performance objective. In this work a rational approach to these problems as well as a practical methodology for doing the acceptability checks for linear and non-linear analysis are presented, being the main objective to answer the following questions :

- How many analysis are needed during the acceptability checks phase of the performance based design ?

- What values should be used for the random variables to model the structure and the EQGMs and other loads that can act simultaneously when doing the acceptability checks?

- How this values should be related to the definition of the performance objective in terms of the target reliability or acceptable failure probability ? ; and

- To prove that in general it is not necessary to carried out a large number of analysis, neither go to upper and lower bounds of all the parameters what can produce an over-conservative response with unknown reliability.

PROBLEM STATEMENT

Assume that it is desired to evaluate if the non-structural damage under the service EQGM satisfy the performance criteria. According with the performance objective **Table 1**, the problem can be formulated as follows :

⇒ Given :

- An EQGM with a return period $T_R = 30$ years
- A building with non-structural components that not suffer visible damage if the maximum inter-story distorsion is $\Theta_{ser} \leq 0.003$

⇒ Required :

- To evaluate if the design satisfies the objective perfomance for non-structural components for frequent EQGMs; i.e., to evaluate if the failure probability P_f is smaller than the performance objective $P_{f_{ser}} = 0.20$

⇒ Solution :

- An efficient solution to this problem requires to obtain an estimation of P_f as close as possible to the real value using as few structural analysis as possible.

SOLUTION

The following notation will be used in order to find a rational solution to this problem :

$Y = \Theta$	Random variable corresponding to the parameter response to be verified.
$X_i\ (i=1,\ldots,n)$	Random variables corresponding to the n data of the problem (Peak ground acceleration, ground acceleration time history $\ddot{u}_g(t)$, size and capacity of the structural elements, accidental eccentricity, gravity load when the EQGM occurs, etc.)
$\mu_Y,\ \sigma_Y$	Mean value and standard deviation of the random variable Y.
$\mu_i,\ \sigma_i$	Mean value and standard deviation of the random variables X_i.
$Y=f(X_1,\ldots,X_n)$	In general using linear and non-linear structural analysis computer programs, the response Y will be obtained for specific values of X_i. The relationship between X_i and Y is represented symbolically here by the function $f(X_i)$.
$Y^* = \Theta_{ser}$	Limit performance objective for the response variable to be evaluated.
$P_{f_{ser}} = \Phi(-\beta^*)$	where, Φ = cumulative standard normal distribution, and β^* = a parameter to measure the target failure probability (For example, for $P_{f_{ser}} = 0.20, \beta^* = 0.84$).

If the random variables X_i are independents, an approximation to μ_Y and σ_Y can be obtained using (Ang-Tang 1975)

$$\begin{aligned} \mu_Y &= f(\mu_1,\ldots,\mu_n) \\ \sigma_Y^2 &= \sum_{i=1}^{n}\left(\frac{\partial f}{\partial X_i}\right)^2_{\mu_i} \sigma_i^2 \end{aligned} \tag{1}$$

Assuming a normal distribution for the random variable Y, an estimation of the failure probability can be obtained as

$$P_f = P(Y > y^*) = 1 - P(Y \le y^*) = 1 - \Phi\left(\frac{y^* - \mu_Y}{\sigma_Y}\right) = 1 - \Phi(\beta) = \Phi(-\beta)$$

$$\beta = \frac{y^* - \mu_Y}{\sigma_Y} \tag{1}$$

The design satisfies the performance objective if $P_f \le P_{f,ser}$ or, $\Phi(-\beta) \le \Phi(-\beta^*)$ or, $\beta \ge \beta^*$. Therefore the design satisfies the performance objective if

$$\frac{y^* - \mu_Y}{\sigma_Y} \ge \beta^* \tag{3}$$

or,

$$\mu_Y \le y^* - \beta^* \sigma_Y \tag{4}$$

Eq.(4) shows that to evaluate the design it is necessary to know μ_Y and σ_Y, the mean value and standard deviation of the response parameter. These values can be obtained from the same computer program used to analyze the structure through the following procedure :

1. To analyze the structure using the mean values of the random variables $x_1 = \mu_1, \dots x_n = \mu_n$. One of the output of the program is the value of the response parameter y. Using eq. (1), an estimation of the mean value of the response can be obtained directly as $\mu_Y \cong y$.

2. To analyze the structure again modifying one of the random variables X_i, using $x_1 = \mu_1$, ... , $x_i = \mu_i + \beta^* \sigma_i$, ... , $x_n = \mu_n$. The analysis is repeated for $i = 1$ *to* n modifying each random variable x_i at a time. In each analysis the response parameter y_i is obtained from the output of the program. An estimation to the response sensibility to a variation in the variable X_i can be obtained as

$$\frac{\partial f}{\partial x_i} \cong \frac{\Delta f}{\Delta x_i} = \frac{f(\mu_1, \dots, \mu_i + \beta^* \sigma_i, \dots, \mu_n) - f(\mu_1, \dots, \mu_i, \dots, \mu_n)}{\mu_i + \beta^* \sigma_i - \mu_i} = \frac{y_i - \mu_Y}{\beta^* \sigma_i} \tag{5}$$

3. To estimate σ_Y using eq.(1) and (5) :

$$\sigma_Y^2 \cong \sum_{i=1}^{n} \left(\frac{\Delta f}{\Delta x_i}\right)^2 \sigma_i^2 = \sum_{i=1}^{n} \left(\frac{y_i - \mu_Y}{\beta^* \sigma_i}\right)^2 \sigma_i^2 = \frac{1}{\beta^{*2}} \sum_{i=1}^{n} (y_i - \mu_Y)^2$$

$$\sigma_Y \cong \frac{1}{\beta^*} \sqrt{\sum_{i=1}^{n} (y_i - \mu_Y)^2} \tag{6}$$

Using eq.(4) and (6) the performance objective is satisfied when

$$\mu_Y \le y^* - \sqrt{\sum_{i=1}^{n} (y_i - \mu_Y)^2} \tag{7}$$

From eq.(7) it is concluded that **for the analysis in the step 2) is only necessary to consider the variation in those variables X_i that can produce large values of $(y_i - \mu_Y)$, i.e., those variables that can produce the largest effects on the response parameter studied.**

In the particular case in which the variation in one of the variables $\Delta X_i = \beta^* \sigma_i$ produce an effect on the response much larger than that produced by the variation in any other variable, from eq.(7) the performance objective is satisfied when

$$\begin{aligned} \mu_Y &\leq y^* - (y_i - \mu_Y) \\ y_i &\leq y^* \end{aligned} \tag{8}$$

Therefore, **in the case in which the variation in one of the variables $\Delta X_i = \beta^* \sigma_i$ produce an effect on the response much larger than that produced by the variation in any other variable, only *one* analysis is needed to evaluate the performance objective**. This analysis must be done using the mean value of the random variables ($x_k = \mu_k$) for $k \neq i$ and the value $x_i = \mu_i + \beta^* \sigma_i$ for the variable x_i. In order to evaluate the required performance objective in this particular case, the response, y_i , obtained from the output of the computer program used for the structural analysis can be directly compared with the target performance value, y^* .

In earthquake engineering it is not unusual that the variation in one random variable has much larger effect on the structural response than the other variables. There are two reasons :

a) because the uncertainty in that variable is much larger than in the others (for example, in case of linear response when the design is based on the response spectra, the PGA and/or the frequency content of the GMs), or

b) because the effect on the structure is extremely sensitive to that variable (for example, the acceleration time history $\ddot{u}_g(t)$ in the case of the non-linear response).

In the first case the structure should be verified with $PGA = \mu_{PGA} + \beta^* \sigma_{PGA}$ (i.e., the mean PGA expected for the return period considered plus β^* times the standard deviation). In the second case, since $\ddot{u}_g(t)$ is a random process the following procedure is recommended : using an equivalent SDOF system and the data base of EQGMs used for design determine which of those EQGMs produce the maximum global response of the local parameter to be studied (displacement, acceleration, ductility demand, energy dissipation demand) ; then use the EQGM selected for each response parameter for verifying the performance objectives using eq.(8).

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NONLIN: A Computer Program for Earthquake Engineering Education

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SUMMARY

The practice of earthquake engineering requires a solid background in structural dynamics, and a mastery of the concepts of inelastic behavior, ductility, and energy dissipation. A thorough understanding of the relationship between structural response and ground motion characteristics is also required.

While the traditional master's degree program in earthquake engineering includes some background in these topics, it is difficult for the student to obtain a clear understanding of the nature of inelastic behavior. One reason for the difficulty in learning is the mathematical complexity of the problem being solved. Since no closed form solution exists for most nonlinear earthquake problems, the student must resort to the study of existing solutions, develop computer programs for performing the analysis, or seek out currently available software.

As a Ph.D. candidate studying under Professor Bertero, the writer was faced with the above dilemma when trying to solve a homework problem in the professor's earthquake engineering course. As part of the homework solution, a simple program for the step-by-step analysis of a single degree of freedom inelastic system was developed. Over the past few years this simple program, now called *NONLIN*, has been expanded in capability, and reformatted into a Microsoft Windows format. The improvement was primarily motivated by, and funded through the writer's participation as lead instructor for the Earthquake Protective Design portion of the Multi-Hazard Building Design Summer Institute (MBDSI). This course, offered at FEMA's Emergency Management Institute in Emmitsburg, Maryland, is intended for college and university engineering faculty who are interested in expanding their capabilities in hazard mitigation design.

The purpose of this short paper is to describe the *NONLIN* program as it currently exists. New features that will be added to the program in time for the July 1997 offering of the MBDSI will also be mentioned. *NONLIN* is currently available, free of charge, to any person with access to the Internet. The program may be downloaded from the following site:

www.fema.gov/emi/nonlin.htm

<u>Program Overview</u>
In NONLIN, the structure is modeled as a single degree of freedom system with a bi-linear force-deformation relationship. The behavior may be linear elastic, elastic-perfectly plastic, or elastic-

plastic with a positive or negative secondary stiffness. The structure may be subjected to an acceleration time history acting at the base, or to a linear combination of sine, square, or saw-tooth forcing functions applied directly to the mass. The structure may also be put into free vibration.

Program Input

The main input screen for *NONLIN* is shown in Figure 1. The structure is depicted as a one-story one-bay frame with a simple linear viscous damper. The single degree of freedom is the lateral displacement of the mass relative to the ground. The five independent structural properties; mass, damping, initial stiffness, secondary stiffness, and yield strength are entered interactively by pressing the appropriate icon. Mass may be entered in either mass or weight units, and damping may be entered as a force/velocity coefficient, or as a ratio of critical damping. At any time during the data input or subsequent analysis the units may be changed by selecting the Unit Type, Length Units, or Force Units options.

The program comes with at least twelve earthquake accelerograms, and new motions may be easily added. These time histories are accessed through the QuikQuake menu, or by pressing the Dynamic Force Applied as Ground Acceleration icon. If the icon is pressed, a new window opens which allows the user to modify the duration, amplitude, and time step digitization of the ground motion. For any earthquake selected, time history plots of ground acceleration, velocity, and displacement may be obtained. The user may also plot a traveling Fourier amplitude spectrum of the motion, or may obtain a wide variety of linear earthquake response spectra plots including tripartite plots, independent log-log, log-arithmetic, or arithmetic-arithmetic plots. *NONLIN*'s response spectra plots are obtained through the piece wise exact integration technique (Wilson, 1996). Earthquake demand spectra may also be plotted, and if desired, the bi-linear structural capacity plot may be superimposed to produce a demand-capacity plot. Each of the plots may be directed to the printer. If desired, blank tripartite graph paper may also be printed.

As an alternate to earthquake ground motions, the structure may be subjected to a linear combination of up to five sine, square, or triangular waves. Each wave form may have a separate frequency, amplitude, duration, or phase lag. Simple one-component waves may be obtained through the QuikWave menu option. The more complicated wave forms are obtained through the Dynamic Force Applied as Forcing Function icon. In the latter case, time history plots and FFTs can be obtained of the wave form. The wave may be applied directly to the structure's mass as a forcing function, or to the base of the structure as an acceleration time-history.

Solution Technique

The single degree of freedom equation of motion is integrated step-by-step using the Newmark constant acceleration technique (Clough and Penzien, 1993). The accuracy of the solution may be monitored by computation of the structural energy time history (described below). If required, the solution may be improved by subdividing the integration time step.

Program Results

Once all the data is input, the time-history analysis is performed by a simple click of the mouse. If some of the input is missing, the program will warn the user and prompt for the required data.

Once the analysis is complete, the results may be viewed by pressing one of the following icons:

 Summary of Results Icon. When pressed, a single page summary of peak response values from the current analysis is displayed. Results from previous analyses may also be viewed. Any of the single-page summaries may be printed.

 Time History Results Icon. When pressed, three separate time-history plots are displayed. Initially, the structural displacement, elasic-plastic resisting force, and yield codes are plotted. These may be changed to display velocity, acceleration, relative inertial force, damping force, elastic-plastic resisting force plus damping force, or elastic-plastic resisting force plus damping force plus relative inertial force. For any time history except yield codes, a traveling FFT may be obtained. Printer plots can be produced, and if desired, a tab-delimited file of the entire analysis may be saved to file for further processing by a separate spreadsheet program.

 X-Y or Hysteresis Plots. When pressed, three separate X-Y plots are displayed. Initially these are elastic-plastic resisting force versus displacement, damping force versus displacement, and relative inertial force versus displacement. The plot types can be easily changed. For example, plots of relative inertial force versus acceleration, and damping force versus velocity can be displayed to illustrate the linear nature of these components of the structural response.

 Energy Time Histories Icon. When this icon is pressed, a time history of the structural energy is displayed. Kinetic energy, strain energy, viscous damping energy, and structural hysteretic energy are shown in separate colors, as well as is the independently computed total input energy. Discrepancies between the total internal and external energy are clearly displayed on the plot. If the total energy from the individual components is not equal to the total input, the user should rerun the analysis using a smaller time step. The energy computations can be performed on a total or relative basis (Uang and Bertero, 1990).

 Animation Icon. One of the most unique features of *NONLIN* is the ability to animate the dynamic response. While the animated structural response is being shown the corresponding time histories and X-Y plots are also displayed. For earthquake problems the displacement relative to the ground, or the total displacement, including the ground displacement may be animated. While the structure moves, the ends of the columns change color whenever a yield event is detected.

Planned Updates to *NONLIN*

In the very near future, the ability to separately plot code-based and Newmark elastic and inelastic design spectra will be added. Automatic computation of true inelastic spectra will also be allowed. This update will be complete by July of 1997.

A more sophisticated two degree of freedom structural model is also planned. The new model, designed for assessing the performance characteristics of structures with passive energy systems will have an improved set of hysteretic energy mechanisms (e.g. nonlinear viscous, friction, and metallic yielding) which may be used separately or added together to produce compound elements. This element will be in series with a variable stiffness chevron brace. This major program update will not be complete until late 1997.

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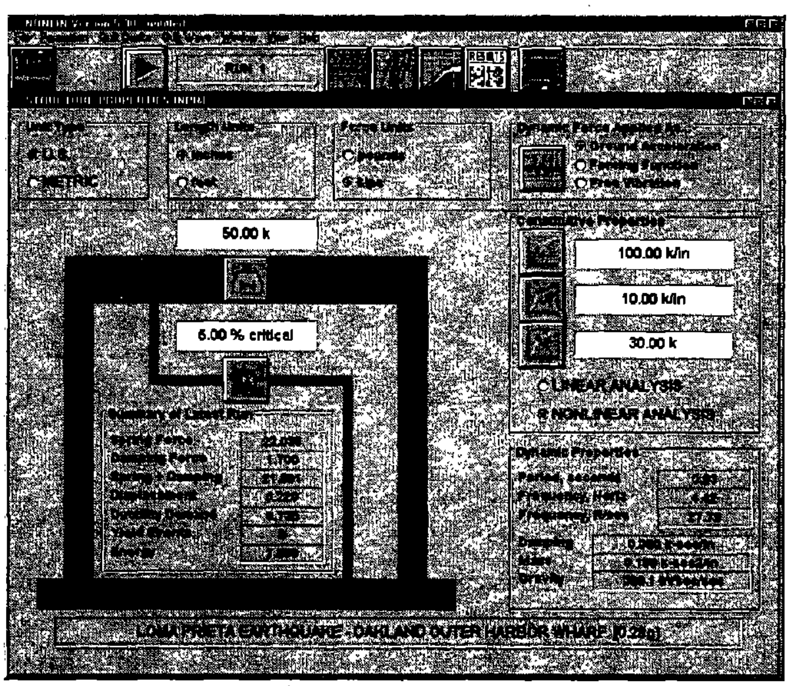

Figure 1. The main *NONLIN* Input Screen

An Evaluation of the Effects of Long Velocity Pulses on the Seismic Response of Structures

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SUMMARY

Three typical chilean R/C structural wall buildings are analyzed considering the effect of long acceleration and velocity pulses. It is shown that main non linear incursions are closely related to long pulses of earthquake ground motions. The maximum energy input in a critical cycle is an additional aspect to be considered in the evaluation of seismic destructiveness. In adition, the building drift ratios are found also closely related to the ground velocity history. Duration of the strong motion appears to be important only in the case of several long aceleration pulses.

INTRODUCTION

Several approaches have been proposed for evaluating seismic destructiveness such as Earthquake Energy Input , E_I (Bertero), Destructive Potencial Factor, P_D (Araya and Saragoni), the Park and Ang's Damage Index, $(DMI)_{PA}$, the Fajfar's dimensionless parameter γ, and the Rodriguez Parameter, I_D. Absorbed energy and maximum drift are the most important parameter considered in some of these approaches. The goal of this paper is to show that the maximum energy input in a critical cycle is an aditional aspect to be considered in the evaluation of seismic destructiveness.

Bertero (1992) has illustrated the importance of one or several long acceleration pulses on seismic response. This important aspect has not been explicitilly included in the above mentioned parameters. Saragoni's and Rodriguez's parameters have produced the best correlations between observed and calculated seismic damages. However, for the P_D parameter the structure characeristics have not been considered and the long acceleration pulse content has been taken into account indirectly through the number of zero crossings in the accelerogram. By other hands, Rodriguez includes the total estimated histeretic energy and maximum displacements, not considering the rate of energy.

In this paper the seismic response of three typical chilean R/C structural wall buildings is analized considering the effect of long acceleration and velocity pulses. It is shown that main non linear incursions are closely related to long pulses of earthquake ground motions. In addition, the building drift ratios are found also closely related to the ground velocity history.

NUMERICAL RESULTS

A typical plant and elevations of a 10 story R/C structural wall building designed according to the current chilean Code (1993) and ACI 318 Code (1989) are shown in **Figure 1**. Three levels of ideal lateral strength were considered for this building. The corresponding buildings have been identified as Building 1 (on firm ground), Building 2 (on soft ground) and Building 3 (walls having twice the flexural strength of corresponding walls in Building 2). The ideal maximun lateral strength were 14, 23 and 36 per cent of the total building weight.

The left side of **Figure 2** shows the six earthquake ground motion records considered in this study and the top building displacement histories obtained from the performed nonlinear analysis. Zooms of the most important non linear incursions are also shown in the right side of **Figure 2**. Top building displacement and ground velocity histories are compared in **Figure 3** in the same time intervals of **Figure 2**. Building seismic responses were calculated using both, Drain-2D (Powell et al, 1993) and Ruaomoko (Carr, 1996) nonlinear analysis programs, which lead to similar global seismic responses.

A global evaluation of results shown in **Figures 2** and **3** can be summarized as the follows:

In regard to top building displacements, Lateral Strength appears to be important only in the Mexico and Northridge records, both having long duration acceleration and velocity pulses.
- Maximum top building displacements occurred inmediatly after the most important acceleration pulse. After this maximum response, in most cases a free dumped vibration decay is observed, especially in building 3 subjected to chilean records (Viña del Mar and Llolleo, 1985).
- Top building displacements and ground velocity histories appear to be approximately in phase, with a better correlation for the Sylmar, JCM and Viña del Mar records, in the range of response close to the first long pulse.

CONCLUSIONS

Maximum seismic response occures due to the incursions of long velocity pulses. After these incursions the structure responds approximately as in the case of free vibration decay until a new incursion due to a long pulse. Only few important nonlinear incursions have been observed in the analysis of typical chilean records, and all of them were related with relatively long acceleration and velocity pulses. Longer pulses are present in the other analyzed records, with greater demands on the structure. Duration of the strong motion appears important only in the case of several long acceleration pulses, as in the Mexico City case. Future research should take into account this characteristic.

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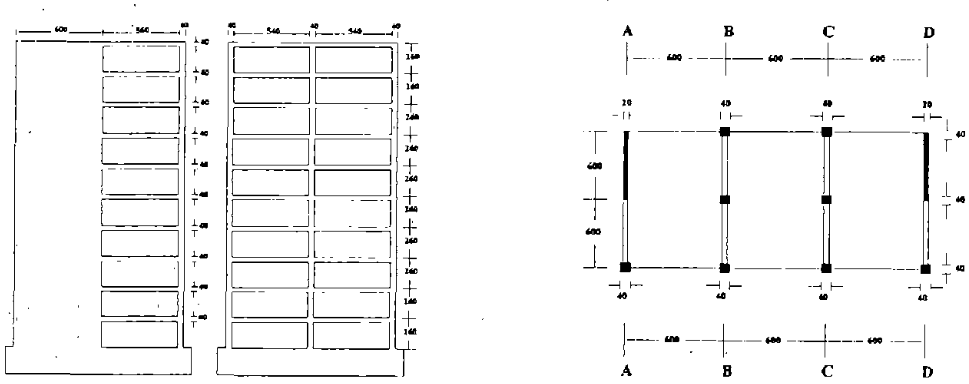


Figure 1. Elevation and Plant Building.

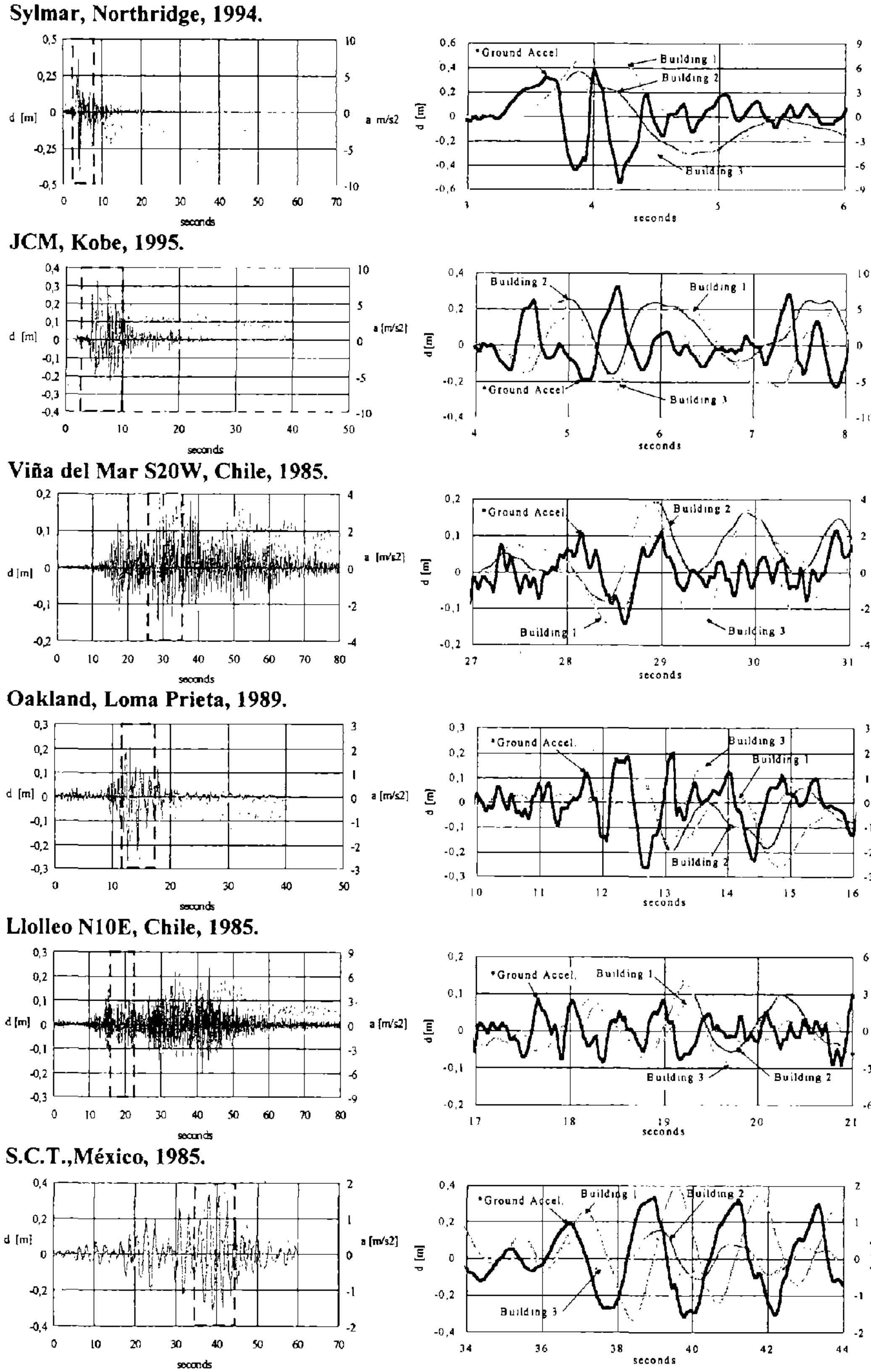


Figure 2. Top displacement and ground aceleration histories.

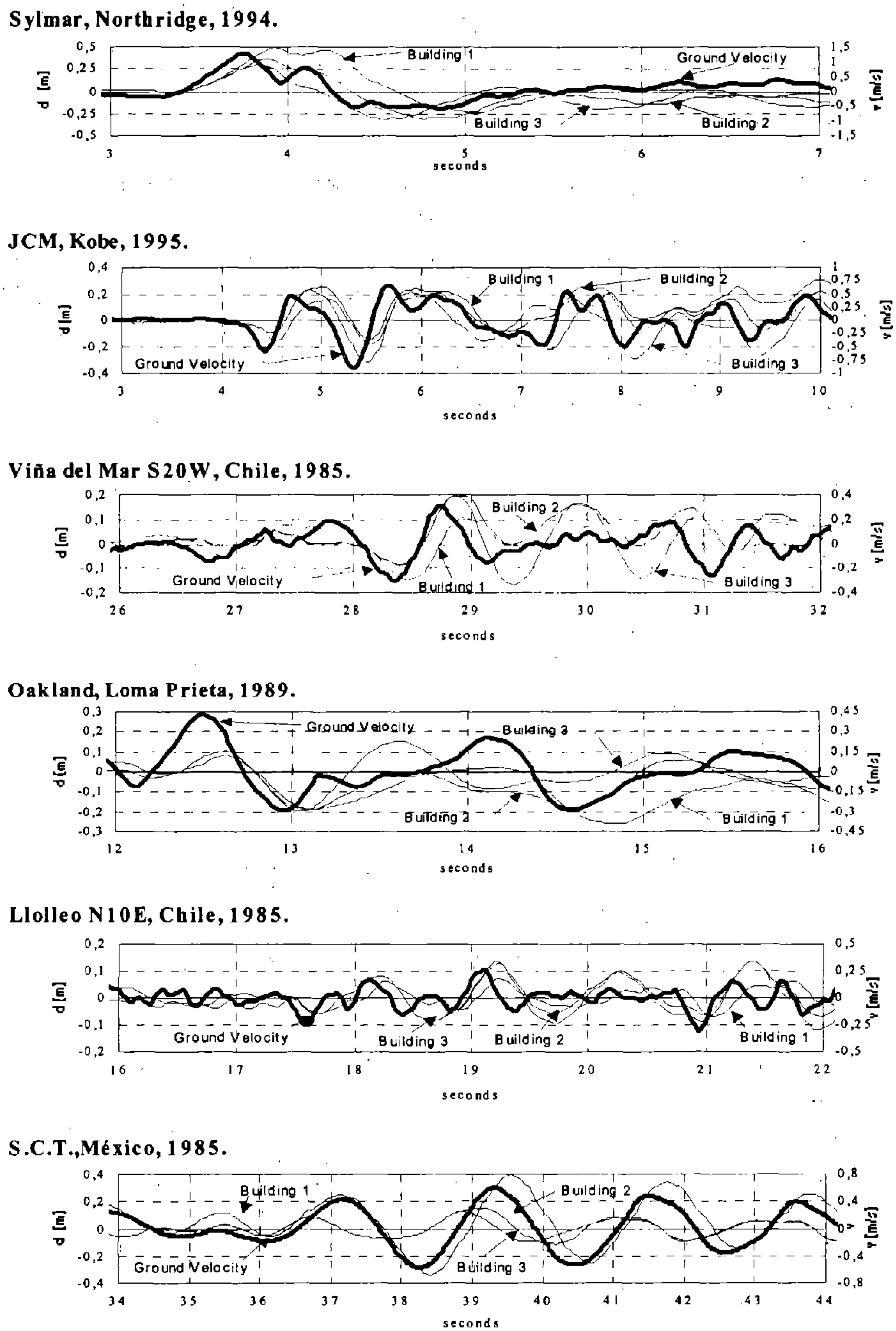
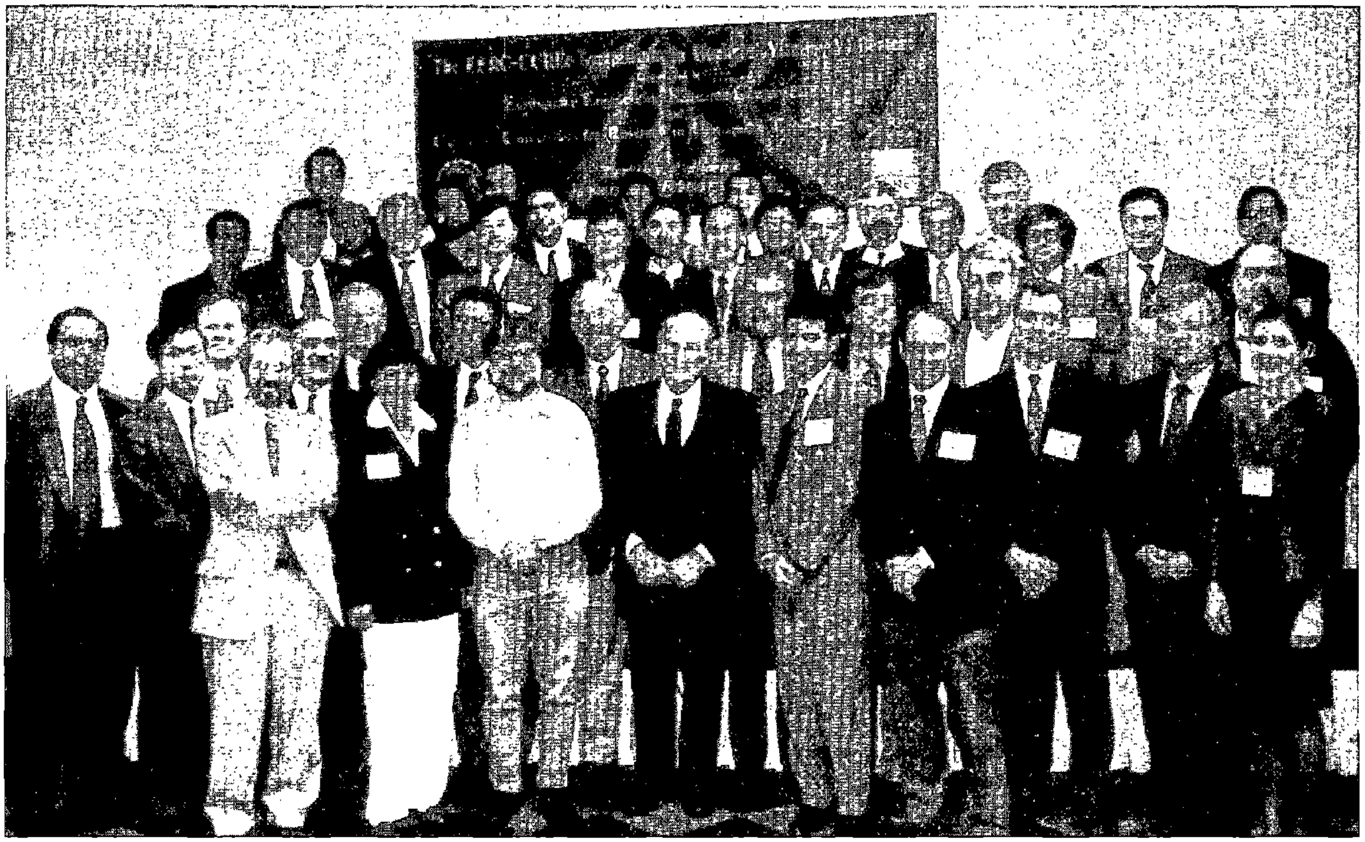


Figure 3. Top Lateral displacement and ground velocity histories.

The essence of the man, Vitelmo Vittorio Bertero.

He never stays at home when earthquakes strike,
and travels all around the globe with many cameras and few assistants.
He preaches that soupply should top δemand,
to comprehensively design, in other wor,
with good δetailing of the steel, confinement of the concrete.
He is possessed with seismic hazard mitigation
but has no qualms about his students' strength deterioration.
He has no fear of earthquakes, because
in spite tremendo mass, he is endowed with stiffness and douctility.
He is a man of many friends, because,
in spite of tough δemands, he is endowed with kindness and humility.

Filip Filippou



Some of Professor Bertero's Former Students

Speakers at the Bertero Symposium
Photos by Barbara Mauk/EERC

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