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ANN ARBOR, MICHIGAN 48109

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16. Abstract (Limit: 200 words)

Papers delivered at the seminar are presented. Among the topics addressed are the development of retrofit guidelines for highway bridges; the seismic resistance of interior partitions; the epoxy repair of concrete components under fire exposure; the repair and strengthening of masonry; the effect of infills in seismic resistant buildings; and soil modification to reduce the potential for liquefaction. It is concluded that the addition of infills introduces significant changes in the dynamic characteristic of the bare moment resisting frame. It modifies the modes of vibration and the damping of the specimens. The stiffness of buildings is found to increase significantly when all the frames are infilled. Solid bricks reinforced externally with welded wire fabric are concluded to offer considerable advantage over the use of the bare frame. Seminar participants are listed and the agenda is provided.

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R.D. Hanson, /PI

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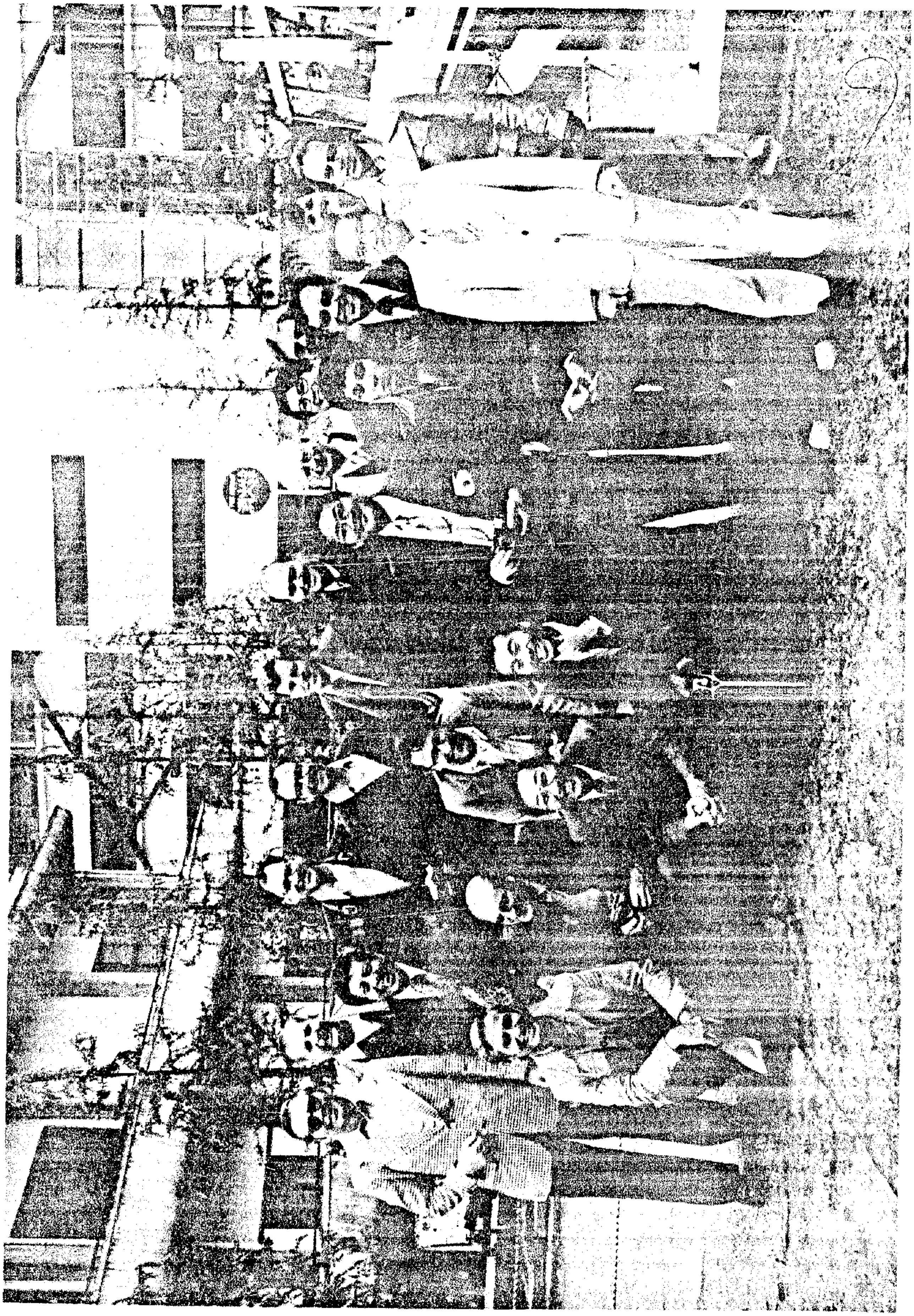

PREFACE

Research, design and construction activities in the repair and retrofit of structures for earthquake resistance both in Japan and the United States have been increasing rapidly over the last decade. One way to maximize the benefits of research and experiences of others is to share them at an early stage of development and discuss alternative approaches and techniques. This was the purpose of the US/Japan Co-operative Research Program in Earthquake Engineering on Repair and Retrofit of Structures sponsored by the National Science Foundation through grant number CEE–7816730 to The University of Michigan.

A series of three seminars (May, 1980 in Los Angeles; May, 1981 in Sendai and Tsukuba, Japan; and May, 1982 in San Francisco) were held to share and discuss research results and field experiences. The Proceedings of these three seminars have been published in three volumes. A fourth volume contains an English translation of several Japanese reports on evaluation of earthquake resistance of existing buildings prepared for Shizuoka Prefecture as part of their Earthquake Hazard Reduction Program.

The financial support of the National Science Foundation, and the personal efforts by Dr. John B. Scalzi, NSF Program Manager, in establishing this program; the contributions of Mihran S. Agbabian and James Warner in organizing the Los Angeles meeting and field trip; and the contributions of Loring A. Wyllie, Jr. and Oris H. Degenkolb in organizing the San Francisco meeting and field trip are sincerely appreciated. The meeting and field trip in Japan was organized by Dr. Makoto Watabe and by Dr. Masaya Hirosawa who receive the sincere thanks and appreciation of all US participants.

The opinions, findings, conclusions and recommendations expressed in these volumes are those of the individual contributors and do not necessarily reflect the views of the NSF or other private or governmental organizations.

Robert D. Hanson
Ann Arbor, Michigan

May 1981, Contents

V

US/Japan Cooperative Research Program in Earthquake Engineering
on Repair and Retrofit of Structures

Proceedings of the Second Seminar—May 1981

TABLE OF CONTENTS

Sunday, May 17, 1981

9:00 a.m. - Leave Hotel

9:00-12:00 m. - Visit to Tohoku Institute of Technology (Micro bus) (Explanation by slides and discussion at T.I.T.)

12:00- 1:00 p.m. - Group Lunch

1:00-4 p.m. - Visit to Izumi High School (Micro bus)

5:19 p.m. - Leave Sendai (Express "Tokiwa 16")

10:08 p.m. - Arrival at Tsuchiura (Micro bus)

10:50 p.m. - Arrival at Tsukuba Hotel (Kenshu Kaikan)

Schedule for Seminar at BRI, Tsukuba

Monday, May 18, 1981

9:00 a.m. - Opening Session - H. Takebayashi, R. D. Hanson

9:10 a.m.-12:30 p.m. - Session I Co-Chairmen: M. Hirosawa, R. D. Hanson

Development of Retrofit Guidelines for Highway Bridges by J. D. Cooper

Introduction to an Earthquake Evaluation Test for Effects to Retrofit of Reinforced Concrete Bridge Pier Elements by S. Kobayashi

Considerations for Retrofitting Bridges by O. H. Degenkolb

Development of Post-Earthquake Measures for Buildings and Structures Damaged by Earthquakes by E. Kuribayashi

Criterion on the Evaluation of Seismic Safety of Existing Reinforced Concrete Buildings by M. Hirosawa

Seismic Resistance of Interior Partitions by M. S. Agbabian

Experimental Studies on Retrofitting Reinforced Concrete Structural Members by Y. Higashi

Epoxy Repair Concrete Components under Fire Exposure by J. M. Plecnik

Discussion

Lunch

1:00 p.m. - Visit to Test Laboratory (S. Okamoto) Full-scale test specimen of seven-storied building US-Japan Cooperative Research Program

INTRODUCTION

The second joint meeting of the US/JAPAN Cooperative Research Program in Earthquake Engineering on Repair and Retrofit of Structures was held in Sendai and Tsukuba, Japan from May 16 through May 18, 1981. This meeting was held immediately preceding the Thirteenth UJNR Panel on Wind and Seismic Effects meeting in Tsukuba. This schedule permitted the attendance of more US governmental representatives than would be possible for a separate meeting.

Fourteen US members and five Japanese members participated in the study tour to Sendai on May 16 and 17, 1981. The tour schedule is summarized later as part of the PROGRAM. Two retrofitted bridges (Sendai-Ohhashi, Abukuma-Ohhashi) and two retrofitted school buildings, which were damaged by the 1978 Miyagi-ken-oki earthquake, were studied and detailed explanations were given by the persons concerned with the repair and retrofits.

a. Both Sendai-Ohhashi and Abukuma-Ohhashi were strengthened by an increasing sectional area of piers with reinforcing bars, injecting epoxy adhesives in the cracked sections and strengthening damaged supports with additional concrete.

b. Building No. 6 of Tohhoku Institute of Technology was strengthened by reinforcing damaged columns, disconnecting spandrel walls at the columns, by constructing steel cross braces in the longitudinal direction, and by constructing new shear walls in the transverse direction.

c. Buildings of Izumi High School were strengthened by reinforcing damaged columns and by constructing new shear walls.

Nineteen US members and twenty-six Japanese members attended a one day technical seminar held at the Building Research Institute in Tsukuba on May 18, 1981. Sixteen papers were reported and thoroughly discussed as detailed in the PROGRAM.

PROGRAM

Schedule for Field Trip to Sendai

Saturday, May 16, 1981

- 7:30 a.m. - Leave Hotel for Ueno Station
- 8:33 a.m. - Leave Ueno (Super Express "Hatsukari 3")
- 12:48 p.m. - Arrival at Sendai
- 12:50 p.m. - Visit to Abukuma-Ohhashi and Sendai-Ohhashi (Explanation by slides and discussion at Tohoku Bureau, Ministry of Construction)
- 5:00 p.m. - Arrival at Hotel (New City Hotel)
- 7:00 p.m. - Group Dinner

c. Information exchange through the seminar is considered to be very important to maximize our knowledge within limited budgets.

d. The next meeting of this cooperative exchange should be scheduled for Hawaii in 1982.

PARTICIPANTS

List of Participants to the Second Joint Meeting

(tour : ○ , meeting : ●)

(U.S.A.)

M.S.Agbabian		●	Agbabian Associates
V.V.Bertero	○	●	University of California at Berkeley
J.D.Cooper	○	●	Federal Highway Administration
O.H.Degenkolb	○	●	Highway Department, State of California
G.F.Fuller	○	●	Department of Housing and Urban Development
R.D.Hanson	○	●	University of Michigan
L.F.Kahn	○	●	Georgia Institute of Technology
H.S.Lew		●	National Bureau of Standards
R.D.McConnell		●	Veterans Administration
J.M.Plecnik	○	●	California State University at Long Beach
J.Warner	○	●	Consulting Engineer. Member of EERI
L.A.Wyllie	○	●	H.Degenkolb and Associates
L.Lund	○	●	Department of Water and Power, City of Los Angeles
C.W.Pinkham	○	●	S.B.Barnes and Associates
L.G.Selna	○	●	University of California at Los Angeles
R.N.White	○	●	Cornell University
J.B.Scalzi		●	National Science Foundation
L.Wang	○	●	University of Oklahoma

ix

2:00 p.m. - Session II Co-Chairmen: J. D. Cooper, S. Okamoto

Guidelines for Retrofitting of Existing Reinforced Concrete Buildings by S. Sugano

Repair and Strengthening of Masonry by L. F. Kahn

Strengthening Effect of Eccentric Steel Braces to Existing Reinforced Concrete Frames by S. Kawamata

Seismic Evaluation and Strengthening of Existing Multistory Residential Buildings by G. R. Fuller

4:00 p.m. - Session III Co-Chairmen: V. V. Bertero, Y. Higashi

Retrofitting of Medium-Rise Reinforced Concrete Housing Structures by M. Hirosawa

Strengthening Existing Concrete and Masonry Buildings for Seismic Resistance by L. A. Wyllie, Jr.

Earthquake Damage at Izumi High School in 1978 Miyagi-ken-oki Earthquake and Methods of Repair and Strengthening by H. Imai

Soil Modification to Reduce the Potential for Liquefaction by J. Warner

Veterans Administration Seismic Correction Program by R. D. McConnell

Effects of Infills in Seismic Resistant Buildings by V. V. Bertero

6:20 p.m. - Closing Session - R. D. Hanson, M. Hirosawa

6:30 p.m. - Group Dinner at Ushiku Chateau

SUMMARY

Thorough discussions followed each presentation to expand and clarify the reported material. It was recommended that the time for the technical sessions be expanded in order to more fully explore common areas of interest in more detail. Because the closing session was no short no formal resolutions were proposed or approved. However, the following specific ideas generated from the meeting were noted.

a. Most structures damaged by recent earthquakes had been repaired and strengthened with or without related analyses. However, most of these works were executed without using standard guidelines.

b. Many existing weak or important structures have been or will be repaired and strengthened according to proposed guidelines.

(JAPAN)

E.Kuribayashi		●	Public Works Research Institute
N.Narita		●	"
R.Hagiwara		●	"
S.Kobayashi		●	"
O.Ueda		●	"
K.Kawashima	○		"
Y.Higashi		●	Tokyo Metroporitan University
S.Kawamata		●	Tohoku Institute of Technology
S.Sugano	○	●	Takenaka Const. Co Ltd.
H.Imai	○	●	Tsukuba University
M.Watabe		●	Building Research Institute
M.Hirosawa	○	●	"
S.Okamoto		●	"
T.Kubota		●	"
A.Shimazu		●	"
H.Hiraishi		●	"
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H.Tsubosaki		●	Goyo Const. Co. Ltd.

X

DEVELOPMENT OF RETROFIT GUIDELINES FOR HIGHWAY BRIDGES

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Federal Highway Administration
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Applied Technology Council
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ABSTRACT

Much recent attention has been given to the development of seismic design guidelines for highway bridges. Unfortunately most existing bridges in the United States have not been designed to resist potentially devastating earthquake induced ground motion. Those existing bridges designed by then state-of-the-art seismic design methods have weaknesses which may require strengthening or retrofitting to reduce the susceptibility to seismically induced damage. Although limited retrofit information is available in the literature, a concerted effort is being made to develop general retrofit guidelines applicable for use in the United States.

This paper highlights vulnerable details associated with existing bridges and presents general retrofit concepts developed to date. A summary of retrofit philosophies developed by several organizations is presented. Finally, issues which must be resolved and will form the basis for the development of retrofit guidelines for existing highway bridges are discussed.

INTRODUCTION

Significant earthquake engineering research has been initiated by many organizations during the past three decades. Structural analyses, designs, building codes, and specifications have been refined and updated to provide the engineer with the necessary tools to design and construct modern buildings to resist the forces developed during periods of strong earthquakes. While research had been conducted to better understand building performance, little research had been conducted in the United States prior to the 1971 San Fernando earthquake to insure the satisfactory seismic performance of highway structures. Since 1971, researchers have focused their attention on the conduct of studies to improve the seismic resistance of both new and existing highway bridges.

The highway network is a vast, sprawling, existing system which forms necessary and vital links between cities and towns across the country. The interstate system of roads extends approximately 41,000 miles and has about 47,000 bridges in the network. Added to that is another 89,000 bridges on the primary system. With very few exceptions, existing highway bridges in the United States have not been designed to resist motions and forces that may be generated by the occurrence of earthquakes in the surrounding areas. As a result, many bridges may be expected to fail in some major way during their remaining life if subjected to strong motion seismic loads. Obviously, the exposure of the network of roads and bridges to seismic hazards varies greatly across the country. Specifically, bridges form the critical links in the road network and are most susceptible to seismic induced damage. They also represent the greatest economic risk if destroyed or damaged.

Two approaches can be taken to improve the seismic resistance of the highway network. One approach requires an investment of time to upgrade seismic resistance while the other requires large sums of money. First, design standards can be upgraded as more knowledge is gained about the response of these specialized transportation structures to seismic activity. New standards can be applied as older bridges are removed from service because they are either structurally unsound or functionally obsolete. This approach, although time consuming, is economically feasible and should be pursued.

The second approach involves identifying those existing bridges which are important to the network and are susceptible to significant damage or collapse in the event of an earthquake. Those structures could be strengthened or retrofit to enhance their response to seismic activity. This approach might prove quite costly and consequently economically infeasible.

What is required is a balanced approach to harden the highway system against seismic attack. This can be accomplished by upgrading those structures which form critically vital links in the network and are vulnerable to damage while at the same time imposing new seismic design standards on bridges which are being replaced.

Specifically this summary paper is intended to focus on the proposed development of a set of retrofit guidelines for use on highway bridges. Vulnerable bridge details are identified. Retrofit concepts previously developed are highlighted and are available in other reports and publications listed in the references. Published retrofit philosophies developed and used by several organizations and countries are summarized.

VULNERABLE DETAILS

Site investigation following earthquakes which cause structural damage and failure has become a necessity to gain insight into failure modes of highway bridges. Based on past earthquake damage investigation and analysis, rational retrofit procedures have and can be developed to insure the structural integrity of the highway system during periods of ground motion. Investigation of numerous earthquakes has pointed to the following general types of damage that most often occur to bridges during seismic attack.

- displacement and tilting of piers
- displacement, cracking and dislodging of superstructure girders
- displacement, settlement and tilting of abutments
- concrete crushing at the supports
- bearing anchor bolt pullout or shearing deformations
- settlement, sliding and tilting of wingwalls
- bearing instability and failure
- expansion joint damage
- settlement of approach slabs

The general types of damage to conventional bridges to be expected in future earthquakes can be grouped into two categories which lead to bridge failure. They are substructure failures (column, pier, or abutment) which lead to loss of support capacity, and superstructure collapse due to excessive relative motion at supports. Both types of failure result from the types of damage documented above. Structural failure and damage to bridges may also be caused by inadequate foundation strength or load-bearing degradation during the course of seismic loading. Soil liquefaction is an example of this failure mode.

Specific retrofit details have been developed based on the observed failure modes mentioned above. Specific details are available in other reports but general concepts developed to date are presented below.

RETROFIT CONCEPTS

Specific retrofit concepts must be based on feasibility and practicality. It is important to emphasize that seismic and structural considerations are not the only ones that need to be considered in the overall bridge retrofit decision process. Other decision factors entering the process are the importance of the bridge to the given locality based on the type of highway, traffic volume and accessibility of other crossings, and replacement or repair costs based on estimated damage including lost time.

A brief summary of some retrofit measures are given below.

- Restrict longitudinal, vertical, and lateral relative displacements of the superstructure at expansion joints and bearing seats, by means of cables, tie bars, shear keys, extra anchor bolts, or metal stoppers.
- Restrict rigid body motion of the superstructure by connecting it with high strength steel cables to a supporting or an adjacent foundation or pier cap, enlarging bearing areas, or placing stoppers at edges of bearing areas.
- Reduce induced vibrations by installation of energy absorbing devices such as elastomeric bearing pads at bearing seats, or adaptation of a "shock absorber" type of damper which allows slow movement such as displacement due to creep, shrinkage, and temperature change with negligible resistance but develops a large resistance in the event of a rapid displacement.

- Strengthen substructure elements such as increasing the strength of an existing column by adding longitudinal and spiral reinforcement to the exterior of the columns, then bonding the added reinforcement with a new layer of high strength concrete using pressure grouting procedures and/or gunite. The added longitudinal reinforcement could also be extended into the cap and the footing thus increasing the flexural strength of the column-to-cap and column-to-footing connections.

There are many variations in detail when actually implementing the retrofit concepts identified above. The designer must be given general guidelines to adapt the concepts to the specific structure in question.

SUMMARY OF BRIDGE SEISMIC RETROFIT PHILOSOPHIES AND EXPERIENCE

There is a very limited amount of published material available that is directed specifically toward the problem of seismic retrofitting of bridges. The California Department of Transportation (CalTrans) has been a leader in the area of bridge retrofitting and is the only State to carry out an extensive construction program. The IIT Research Institute has conducted research on bridge retrofitting for the Federal Highway Administration and two countries, Japan and New Zealand, have published material describing their approaches to the retrofit problem. The following paragraphs summarize the philosophies of each of these four organizations and countries.

California Department of Transportation

Following the 1971 San Fernando earthquake, CalTrans undertook a program to strengthen seismically deficient bridges. Many structures were deficient and it was not economically feasibile to strengthen all of them to new design standards. Efforts were directed toward the most cost effective retrofit concepts. These retrofit concepts consisted of elastic restrainers designed to prevent separation of the sections of a bridge structure at the expansion joints. These restrainers were relatively inexpensive and prevented failure by reducing the chances of support loss at the expansion joints and by distributing forces more uniformly to the columns.

In selecting bridges for retrofitting, a prioritizing system was developed, but final retrofitting decisions relied heavily on engineering judgment. Initially structures on major lifeline routes within densely populated areas were strengthened. Second to be retrofit were those less critical structures located within the same densely populated areas. Finally, those seismically deficient bridges in less populated areas will be retrofitted.

Restrainers are designed to resist a minimum force level based on the weight of the lightest member being restrained. Higher force levels are used if predicted by a dynamic analysis. In many cases several dynamic analyses are performed with different parameters and the results tempered with judgment to obtain the correct design forces. Physical tests were performed to determine the capacity of the restrainers most commonly used by CalTrans.

Column retrofitting to increase available ductility is being considered by CalTrans although no construction has taken place. The schemes under consideration are designed to increase concrete confinement in the area of plastic hinges. Inadequate bond length of main reinforcement is also a problem with some CalTrans structures. Details have been developed but have not been implemented.

IIT RESEARCH INSTITUTE

The Illinois Institute of Technology Research Institute conducted a study on bridge retrofitting for the Federal Highway Administration. In this study three steps of the bridge retrofit decision process were identified and studied. These steps include:

1. A determination of the susceptability of the existing bridge to a critical failure resulting from an earthquake loading.
2. A determination of the level of importance of the bridge to the given locality.
3. A determination of the type of retrofit measures to employ.

To determine the relative degree of vulnerability of a bridge to failure, preliminary assessment of critical structural factors is proposed. Bridges which have a predetermined structural factor value require further analysis to more accurately establish their vulnerability. A simplified analysis method was developed and subsequently modified during the course of the project. This method reduces a bridge to an equivalent single degree of freedom system. Limit states are defined which represent catastrophic failure so that the results of this analysis can be systematically interpreted to establish the vulnerability of the structure.

A method was developed for establishing the criticality or importance of a bridge. A numerical value is assigned to each bridge to reflect its relative importance in terms of administration/transportation systems effects; social/survival effects; security/defense effects; and economic/personal effects. The criticality of a bridge is compared with its structural integrity to determine if the bridge warrants retrofitting.

In determining the type of retrofit measures to be used, the type of failure modes and damage experienced by highway bridges in previous earthquakes was considered. Failures are categorized as either loss of substructure strength and/or stability of excessive relative movement at the bearings. Eight retrofit measures were identified. The analytical determination of forces to be used in the design of these retrofit measures was not covered in the study. In an actual design it was proposed that these forces be determined from a seismic analysis.

JAPAN

The Japanese propose a probabilistic approach to the design of retrofit measures. Bridges are selected for retrofitting based on physical and socio-economic conditions relating to the bridge. It is proposed that these bridges be strengthened to have a reasonably small probability of failure but not to the extent that costs become excessive.

To determine the probability of failure the structural resistance is assumed to be totally deterministic (i.e. there is no variation in dimension or material properties that could cause a probable distribution of resistance levels). Resistance is assumed to deteriorate at a known rate with the increased age of the structure. Therefore, at any point in time, the probability of failure depends only on the probability of a seismic loading that will exceed the resistance level.

Excessive costs will result if retrofit design is based on force levels and failure probability used for new bridges. Therefore, it is proposed to use force levels which will result in a probability of failure consistent with the remaining service life of the structure.

NEW ZEALAND

The draft New Zealand Seismic Design Code for Bridges has a section on the strengthening of existing bridges for seismic loads. This section states that the need for seismic retrofitting should be established by comparing the seismic risk with other risks by any one of several techniques such as a cost-benefit analysis.

The importance of a bridge is established and retrofitting priorities set by considering several socio/economic factors relating to the bridge. New Zealand design force levels are described elsewhere in the code and are dependent on an earthquake return period based on the structure life and importance. The design force levels for retrofitting existing bridges are determined in the same manner as for new bridges except that the remaining economic life is used in place of the new structure life.

Design for smaller force levels are allowed if it is not cost effective to strengthen the structure to the full force level. Designers are cautioned to be aware of the overall behavior of the structure during an earthquake and the effect strengthening measures might have.

Retrofit measures should conform to the same principles of capacity design used for new structures. Design details are not specified, but the designer is instructed to select appropriate details after an adequate site inspection, review of the design calculations and construction drawings, and an analysis. Certain methods used to retrofit bridges are summarized.

DEVELOPMENT OF RETROFIT GUIDELINES

The existing highway system has many bridges which either have not been designed to resist earthquake induced ground motion or have been designed by older, inadequate seismic design standards. Thus it is necessary to focus attention on the seismic protection of those existing structures which are known to be vulnerable to seismic attack. To accomplish that objective, the Federal Highway Administration awarded a contract to the Applied Technology

Council (ATC) to develop seismic retrofitting guidelines for highway bridges. Representative segments of the bridge design and construction profession form a Project Engineering Panel (PEP), Appendix A, and are participating in the development of the guidelines.

Seven issues have been raised by the ATC staff and are to be considered by the PEP to define the scope and general format of the proposed retrofit guidelines. Summaries of the seven issues follow. They include a question followed by a brief discussion of the issue. Answers to the questions have not been finalized and are presented herein for the purpose of stimulating additional thought and discussion. Undoubtedly additional issues will be raised throughout the development of the guidelines.

ISSUE 1 - What aspects of the bridge retrofit problem should be addressed by the guidelines?

The bridge retrofitting problem can be divided into two major areas of concern. The first deals with the evaluation of the seismic resistance of existing bridges and the selection of bridges to be retrofitted. Since it may not be economically feasible to retrofit all seismically deficient bridges to provide earthquake resistance equivalent to new bridges, the selection process is important if the best use of resources is to be realized. The second major area of concern is the design of improvements to increase the seismic resistance of bridges. Because many possible methods of retrofitting are unproven or excessively expensive, partial strengthening should be considered. Selection of the appropriate levels of seismic performance for partially strengthened bridges may depend on the evaluation of the existing bridge. Therefore, it may be difficult to write guidelines that adequately address the design of retrofit measures without including some method of bridge seismic evaluation.

ISSUE 2 - What should be included in the guidelines in relation to the evaluation of existing bridges?

The proper evaluation of the seismic resistance of a bridge depends on the availability of accurate information about the characteristics of the bridge, its location, and seismic exposure. Certain information will be required if a specific evaluation system is to be used. It would be appropriate if the guidelines were to specify the type of information required.

Because a large number of bridges exist in the highway systems of some jurisdictions, it is impractical to perform a thorough investigation, including dynamic analysis, of all bridges in that system. A preliminary screening method is needed to select only the bridges which have the highest potential for failure during an earthquake. A method of performing preliminary screening has been used by CalTrans in their retrofit program. An alternate approach has also been recommended by IIT Research Institute as a part of their investigation of the bridge retrofit problem. A method of utilizing concepts developed by ATC to accomplish the preliminary screening could use the seismic performance category of the bridge to establish importance and seismicity plus the characteristics of structural deficiency developed by CalTrans or IIT Research Institute.

A method of quantitatively rating the seismic hazard of existing bridges could be used as the basis for a benefit to cost analysis for establishing final retrofit priorities or for determining the merits of retrofitting a given bridge to various levels of seismic resistance. In addition, the rating system could provide a method for considering the remaining life of a bridge. One possible rating system would first require an analysis to be performed to determine the effective peak acceleration of a damaging earthquake. The probability of an earthquake of this magnitude occurring at the bridge site within the remaining life of the structure could be obtained from peak acceleration maps and probalistic relationships already developed. The importance of the bridge would be established in terms of a lifeline classification taken from a table similar to a method used to establish the occupancy potential of existing buildings in ATC-3-06 (Tentative Provisions for the Development of Seismic Regulations for Buildings). By multiplying the probability of a damaging earthquake by the lifeline classification, the relative seismic hazard rating of the bridge is obtained. This rating can then be used to make retrofitting decisions.

ISSUE 3 - What type of retrofit concepts should be addressed by the guidelines?

Although many retrofit concepts have been presented in the literature, virtually all retrofit construction to date has dealt with providing elastic restraint at bearings to prevent instability or loss of support. Such restraint also tends to distribute seismic forces in the columns more uniformly, and can thus prevent column failures. Restraint of this type can be a cost-effective method of retrofitting.

Column retrofitting requires complicated details, involves difficult construction procedures, and the relative effectiveness of various retrofit concepts has not been demonstrated by physical testing. Small-scale physical tests of a column retrofit concept using steel banding to increase concrete confinement has demonstrated an increased available ductility but may prove less effective on full-scale bridge columns. Increasing column yield levels for moment may overload the foundation or cause serious column shear failures thus resulting in more harm than good. It would appear that not enough is known at this time to gauge the effectiveness of column retrofitting concepts.

Special energy dissipating bearings and other similar devices have been used extensively in New Zealand and Japan but are virtually nonexistent in the United States. One of the primary reasons for this is the lack of availability of analytical tools to determine the response of structures fitted with these devices. If the analytical tools are made available, the use of these special devices could open up a whole new area in seismic resistant bridge design.

ISSUE 4 - What type of analysis procedures should be used for evaluation and/ or design?

To date, retrofitting of existing bridges has consisted mostly of fitting expansion joints with elastic restrainers to prevent separation and loss of support at these joints. The primary modes of vibration that can cause this type of failure are longitudinal and include both in-phase and out-of-phase vibration of adjacent sections of the bridge. The analysis procedures do not consider different ground motions at various supports or the rotation of the columns due to surface wave effects. Little research information is currently available at this time, but approximate analyses are available.

The use of special energy-dissipating and isolating bearings has great potential in the retrofitting of deficient bridges. These devices, in many cases, rely on their nonlinear behavior to modify the forces and displacements in the bridge. Many of these effects may be difficult to determine with an elastic analysis. Nonlinear analysis computer programs are available, but are difficult and expensive to use. To overcome the need for nonlinear analysis the New Zealand Ministry of Works and Development has developed design charts.

ISSUE 5 - What method of design and evaluation should be specified in the retrofit guidelines?

Seismic design guidelines developed by ATC allow both load factor and working stress methods for design. In evaluating existing structures, however, the ultimate strength of the structure is the principal concern. By using an ultimate strength approach, a more reliable evaluation of the relative danger of structural collapse can be made.

In evaluating existing structures it will be necessary to define the limit states which will represent loss of support or serious structural damage. In the case of structures with substandard confinement in the columns, for example, it will be necessary to account for the reduced available ductility. At bearings, where excessive movement can result in loss of support, a method of accounting for effects such as nonuniform support motion that are not considered in the analysis should be included. The newly developed ATC guidelines use a minimum support length concept based on superstructure length and column height to account for unknown displacement effects. This concept may be inappropriate for evaluating existing bridges. The use of special analysis methods or response modification factors to account for increased displacements and reduced available ductilities may be the best way to approach the problem.

ISSUE 6 - How shall the guidelines address the design of specific seismic strengthening measures?

The design of retrofit measures has problems similar to the design of any type of modification to an existing structure. The designer is restricted in what he can economically accomplish by the characteristics of the existing bridge. In addition, he must be aware of the effect any construction will have on the normal operation of the bridge. To a certain degree, each retrofitting design is unique. Standardization of details can be accomplished only to the extent that the original structural details are standardized. Some standardization has been possible in California, for example, in the design of retrofit measures for intermediate expansion joints in concrete bridges.

Presentation of details, whether standardized or not, is useful since it provides the bridge designer concepts to which simple modifications may be possible.

It is necessary that the designer be given criteria on which to base his design. In the case of expansion joint restrainers, for example, there may be a trade-off between force in the restrainer and displacement of the joint. Allowable capacities should be specified in the guidelines. Since the design of certain retrofit measures may involve a trial-and-error solution, a preliminary design procedure would also be helpful to obtain a realistic first try.

ISSUE 7 - What force levels should be used for the design of retrofit measures?

Based on seismic design guidelines developed by ATC, new structures are designed to resist force levels that have a 10% chance of occurring in 50 years, the assumed economic life of the new structure. Retrofit measures could be designed to resist the same force levels as new structures, but if the remaining useful life of the existing structure is less than 50 years, then the probability of the structure being subjected to these loads is less than that of a new structure. To design the retrofit measures based on equal probability of failure requires that a reduced force level be used for structures with remaining lives of less than 50 years.

From an economic point of view, designing to a specified force level may not be cost effective. For example, in the case of expansion joint restrainers, very little additional cost may be required to increase the capacity of the restrainers to new design standards. On the other hand, there may be a practical limit as to how much additional earthquake resistance expansion joint restrainers can provide. To require design to standards for new structures, or even to a force level based on an equivalent probability of failure may require column strengthening which can have a much smaller benefit-to-cost ratio than joint restrainers. Therefore, it may be justifiable to strengthen several structures to resist smaller force levels than to spend an equivalent amount to strengthen one structure to new standards.

Once the above issues have been resolved, an initial draft of retrofit guidelines will be prepared. They will be refined and updated as more information becomes available and will eventually form a supplement to the seismic design guidelines for new bridge construction developed by ATC.

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INTRODUCTION TO AN EARTHQUAKE EVALUATION TEST FOR EFFECTS TO RETROFIT OF REINFORCED CONCRETE BRIDGE PIER ELEMENTS

by

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1. Introduction

This paper briefly describes an evaluation test for effects to retrofit of reinforced concrete bridge pier elements conducted at the Public Works Research Institute in the past.

In order to increase strengthes of the existing concrete structures there are many kinds of methods in the design and construction procedures as follows,

1). Strengthening using steel plates
2). Strengthening employing Prestress Forces
3). Strengthening casting additional sectional area of reinforced concrete

and 4). Strengthening using additional members of reinforced concrete

In this paper authors introduce an example which evaluated the method 1. and method 3. of the above.

2. Experimental Study

2.1 Specimen Details

Five kinds of test specimens were designed to meet the objectives of this program. Table-1. and Fig.1 show the datails and dimensions of the specimens.

Specimen R is 30cm wide, 30cm deep and 2m high original reinforced column member.

Specimens RR-1,RR-2,RR-3 and RS are retrofitted members.

The method to strengthen for RR's is additional casting of 10cm deep reinforced concrete on both tension and compression sides of the original member R. The differences among RR-1,RR-2, and RR-3 are the method of the pretreatment on the surface to which additional concrete is casted. Details are shown in Table-1.

The strengthening method used for RS is to adhere 6mm thick steel plates on both the tension and compression sides of the original member.

2.2 Material Properties

Material properties used for the specimens were as follows.

Concrete: Ready mixed concrete with high-early portland cement, crashed rock coarse aggregate 20mm in maximam size and river sand was used. The mix proportion of the concrete is shown in Table-2.
The strength of concrete is shown in Table-3.

Steel: Deformed bars used for reinforcement were SD30(JIS Degignation) of 13mm nominal diameter.
Steel plates used for strengthening RS are SS41(JIS Degignation) of 6mm thick.

Adhesive agent: Epoxy resin was used for sticking steel plates, for fixing anchor bars and for spreading on the joint surface of RR-2 and RR-3. Mechanical properties of the resin is shown in Table-5.

2.3 Casting of Specimens

Original column: 10 original columns were fablicated. Concrete was casted continuously being compacted with inner vibrators.

Retrofit 8 columns of original ones were retrofitted with the methods shown in Table-1 and Fig.1.

2.4 Testing Procedure

For giving both the axial force and the bending moment eccentric longitudinal loads were applied by a 1000t compression test machine. The distance of the eccentricy is shown in Table-6.

The load was not increased monotoniously but repeated loading and unloading several times as shown in Table-8 until it reached at the ultimate state. The loading speed was 10t/min.(for R) or 20t/min. (for others). Strains of concrete,reinforcement and steel plates, and the deformation of each specimen were measured during the test.

3. Test Results

3.1 Load-Strain Relations

The relation between the applied load and observed strain of each specimen is shown in Fig.2 and Fig.3. Calculated values in the figures are given by the elastic design method assuming that the ratio of Young's modulus of steel to concrete and the cross sectional area of members are as shown in Table-7.

The relationship between loads and strains of the specimen which was strengthen by additional casting of reinforced concrete is similar to the calculated value when the strength of the load is around the level as usually allowed. In other words the concrete member which casted later is working as a part of the member.

The difference of the treatment of the joint surface selected in this program scarcely affected to the strength of the retrofitted member.

The load-strain relation of the specimen strengthen with steel plates is also similar to the elastically calculated value when the load is such low as the stress of concrete is around the allowable stress which is 100kg/cm² in this case. But when the load exceeds the value, strains of the specimen exceed gradually to the caluculated value and they appeoach to the values which were caluculated negrecting the strength of compression of the steel plate.

3.2 Failure Pattern

Typical examples of failure are shown in Fig.4. In the case of RR type, tention cracks appeared first to the tension side, then vertical cracks were observed in the additional concrete subjected to tensile stresses and lastly a long crack extended along the joint of the compression side.

In the case of RS type, the steel plate on compression side was partly separated from the surface by buckling when the compressive stress of the steel plate reached to around 1300kg/cm². When the load reached a level at 120t, suspected shear cracks appeared at the end of the steel plate subjected to tensile stresses. The load decreased after the compressive failure was observed, when the load reached 140t.

3.3 Bearing Capacities of Loads

The comparison of the observed bearing capacity with calculated one is shown in Table-8. Calc. A is given by the working stress method. Calc. B is given by the ultimate strength method. In case of RS, the strength of the compressive steel was neglected as it had buckled before the maximum load was applied.

The reason that the observed strengths of case RR's were lower than calculated ultimate strengths is that the calculation of strength was carried out assuming that concrete was homogeneous and the strength of the original concrete was assumed as the same strength to additional one although the strength of the original concrete was weaker than the post casted one.

4. Conclusions

Conculusions through the tests and analysis are given as follows.

1). The concrete column member retrofitted by additional casting of reinforced concrete behaves like as a composite structure and its effect is seemed to be reliable.

2). For evaluating the effect of the retrofit by above method, the working stress method can be comprehensively, used.

3). There was little difference on the effect of pretreating of the joint surface among the three methods selected in this program.

4). The retrofitting method to adhere steel plates may cause buckling of the compression steel plate if the compressive stress is subjected. Then,

5). It is recommended to neglect the effect of steel plates as the compression member.

Those tests were carried out under the one directional static load. It is recommendable that the necessity to do more other tests which apply the reversible load for examining whether the conclusions are reliable or not to the bridge pier which would be subjected to such strong reversible load as earthquakes.

5. Acknowledgments

The authors sincerely express their thanks for the coopration of Prof. M.Ohta, Kanazawa Institute of Technology, former head of Concrete Division, the Public Works Research Institute.

Table-1 Specimens

Specimen Type	Retrofitting Method	Steel Origi.	Steel Ret.	Concrete Strength Origin. No.1	Concrete Strength Origin. No.2	Concrete Strength Ret.	Space of tie bar (cm)	Number
R	Original Member.	SD30 D13	–	384	273	–	15	2
RR-1	Chipping. Anchor bar. Additional cast of R.C.	SD30 D13	SD30 D13	384	273	458	15	2
RR-2	Chipping & resin spread. Anchor bar. Add.cast of R.C.	SD30 D13	SD30 D13	384	273	458	15	2
RR-3	Grinding & resin spread. Anchor bar. Additional cast of R.C.	SD30 D13	SD30 D13	384	273	458	15	2
RS	Grind. & resin spread. Sticking steel plates.	SD30 D13	SS41	384	273	—	15	2

Table-2 Mix Proportion of Concrete

Max. Agg. Size (mm)	Slump (cm)	W/C (%)	S/a (%)	Amount in $1m^3$ (kg) W	C	S	G
20	8±2	62	44	175	282	841	1091

Table-3 Strength and Young's Modulus of Concrete

		Comp. Strength (kg/cm^2)	Young's Modulus (kg/cm^2)
Original Concrete	No.1	384	2.6×10^5
	No.2	273	2.1×10^5
Addition. Concrete		458	2.9×10^5

Table-4 Mechanical Properties of Steel

Steel	Yield. Streng. kg/cm^2	Ultimate Streng. kg/cm^2	Elongation %
SD-30 D13	3460	5160	25
SS41	3060	4770	25

Table-5 Mechanical Properties of Epoxy Resin

Factor \ Use for	Spreading on Joint	Fixing Anchor bar	Sticking Steel Plate
Specific Grav.	1.29	1.22	1.17
Flex. Strength kg/cm^2	523	-	780
Comp. Strength kg/cm^2	1219	1031	1946
Young's Modul. kg/cm^2	24900	26500	27800
Tensile Streng. kg/cm^2	266	-	473

Table-6 Loading Process

Specimens	Eccentricity cm	Stress kg/cm^2	Load t	Repetition n
R	12	≒ 100 " 150 " 300	40 60 80	1 1 3
RR	15	≒ 100 " 150 " 200	60 150 210	1 4 5
RS	12	≒ 100 " 200 " 300	60 80 120	1 6 4

Table-7 Assumptions for Calculation

Specimens	Young's Mod. Ratio	Assumption
R	9.0	
RR	7.0	Strengthes of the concrete are equal to the additional concrete.
RS	9.0	Caluc.1 Compressive strength of steel plate is effective. Caluc.2 Compressive strength of steel plate is ineffective.

Table-8 Bearing Capacities

Specimen		Observed t	Calcu. A t	Calc. B t
R	No.1 No.2	110* 97	99 71	124 95
RR-1	No.1 No.2	268 250	247	295
RR-2	No.1 NO.2	210* 260		
RR-3	No.1 NO.2	260 260		
RS	No.1 No.2	140 140	123 88	149 113

A.: The Working Stress Method.
B.: The Ultimate Strength Method.
* : Locally failed at the loading point.
In all of the cases, the compressive side yielded first.

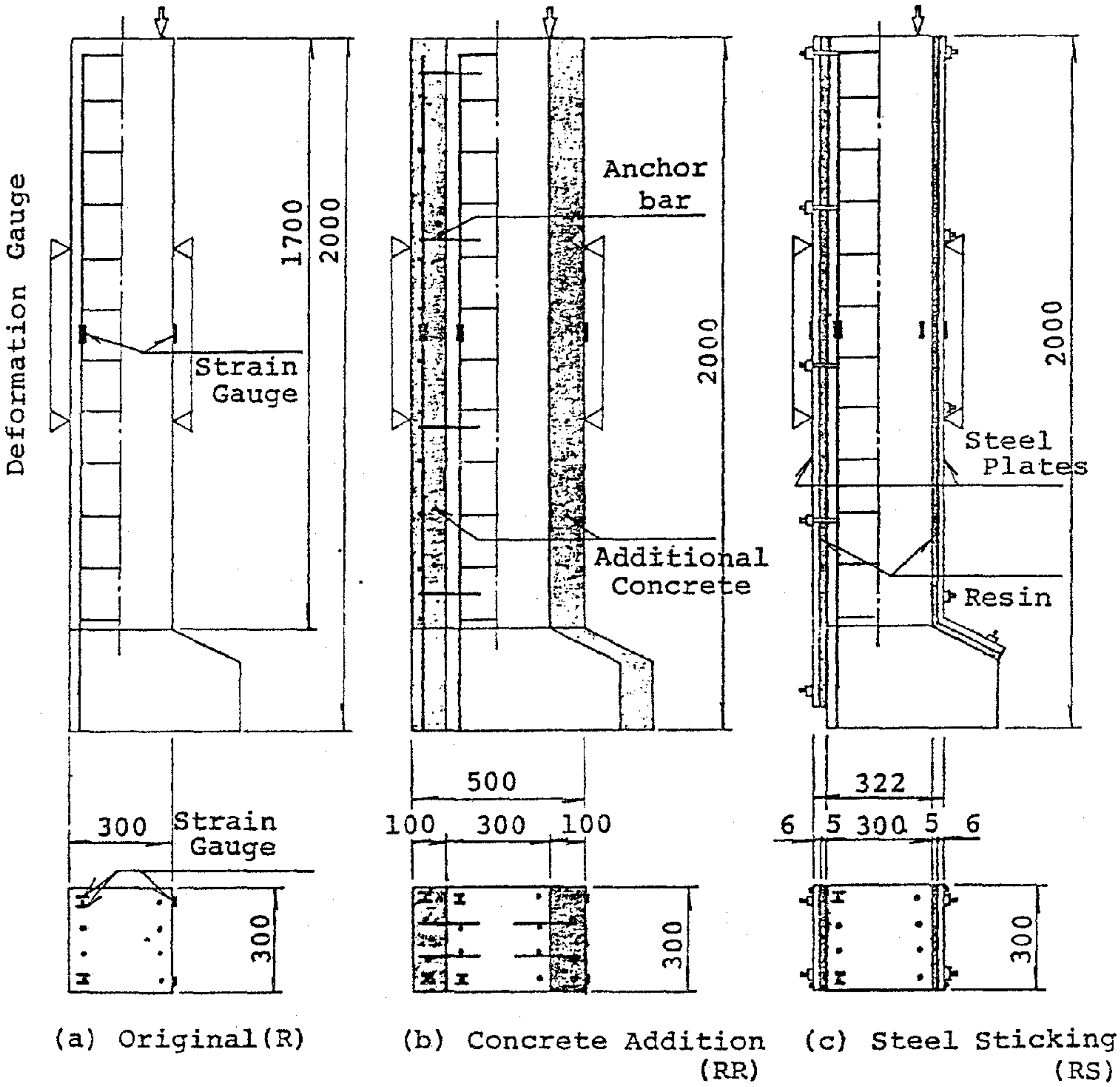


Fig.1 Dimensions of Specimens

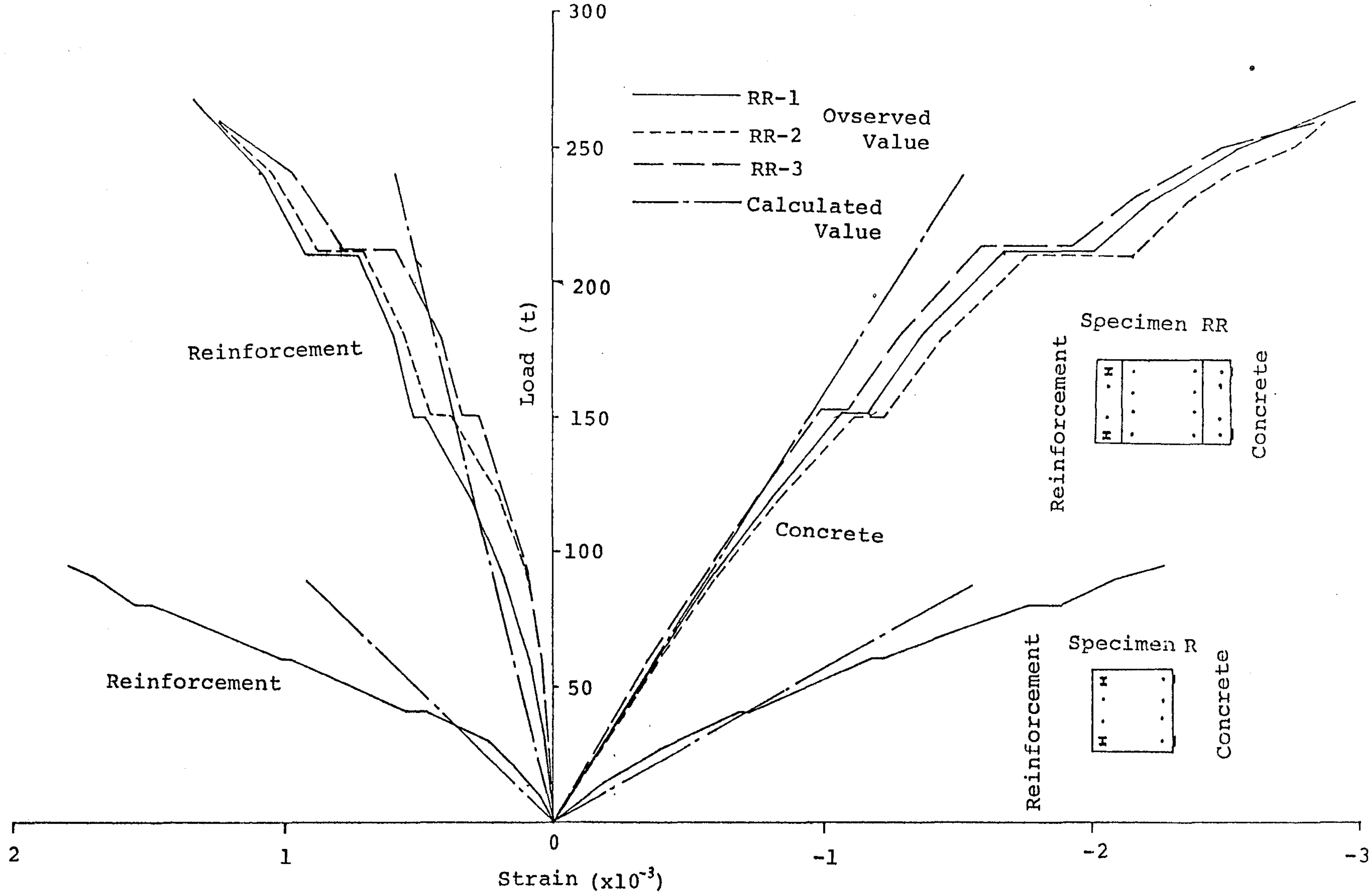


Fig.2 Load-Strain of R & RR

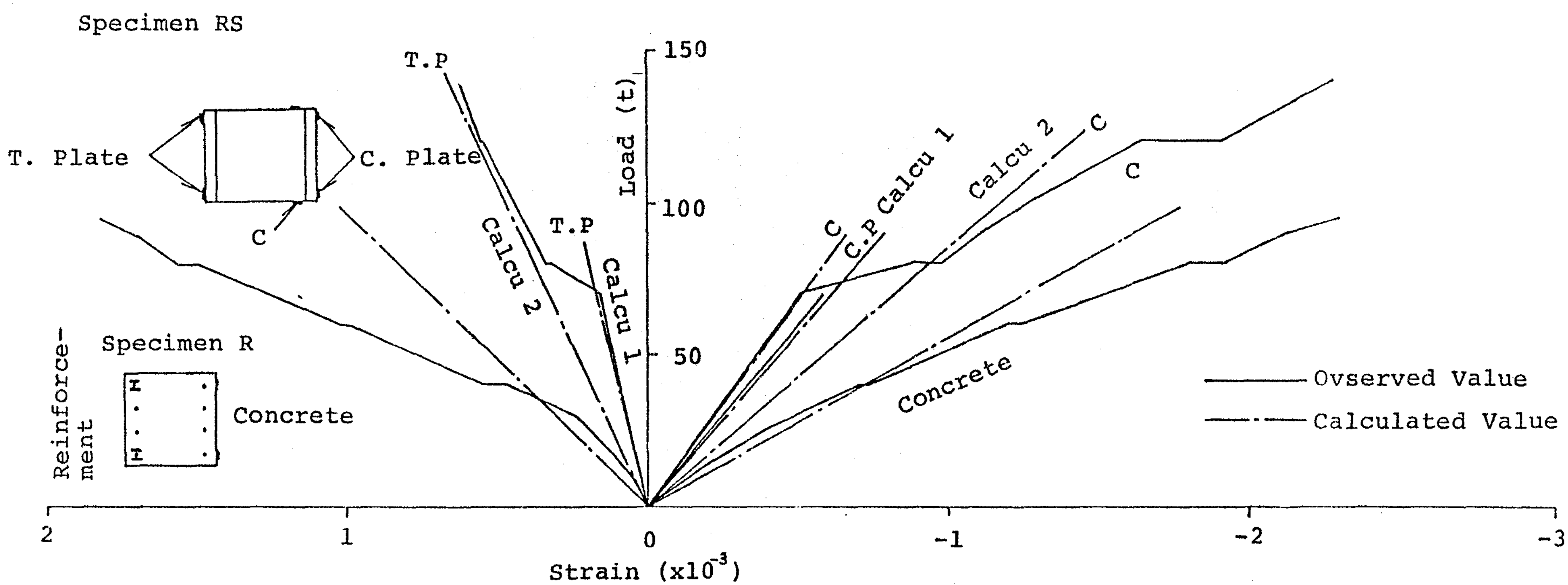
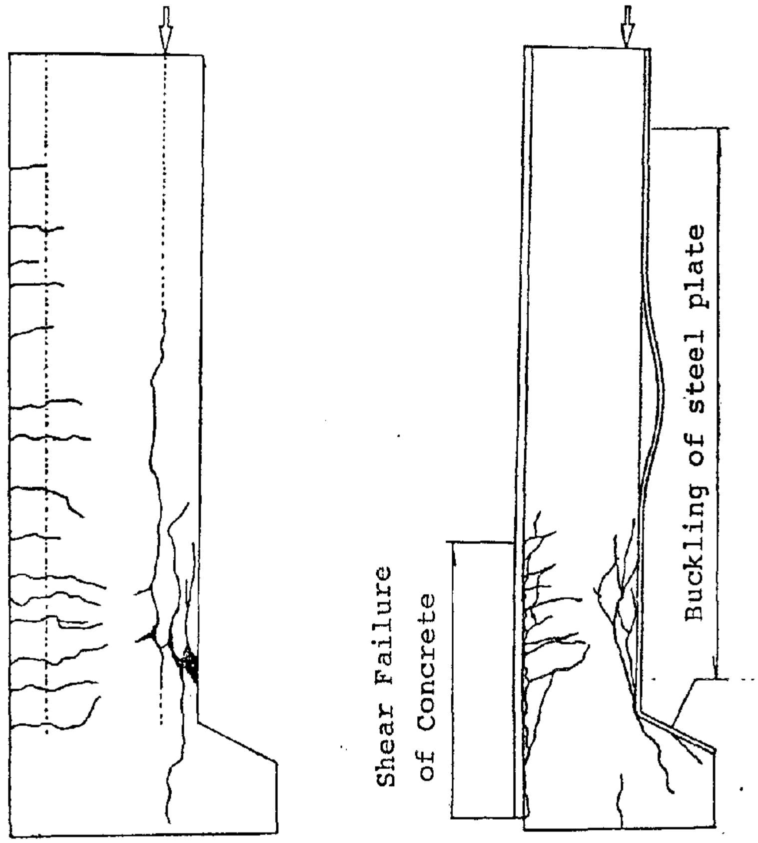


Fig.3 Load-Strain of R & RS



Specimen RR Specimen RS

Fig.4 Failure Mode of RR & RS

CONSIDERATIONS FOR RETROFITTING BRIDGES

Oris H. Degenkolb

May 1981

Introduction

The San Fernando earthquake of February 9, 1981 was the only event to cause any significant amount of damage to any of California's bridges. The total amount of earthquake damage to bridges experienced before that time was minor and was generally ignored. Five small earthquakes since 1971 have caused some minor damage and the collapse of two spans of one four span bridge. The knowledge gained by studying that damage gives an insight into how structures react to seismic shaking and what can be done to mitigate the damage expected from the larger earthquakes that are certain to occur in the future.

Studying the damage from minor earthquakes is valuable because it demonstrates the stages of failure and it is not necessary to speculate on the sequence of events as might be done when conducting a post-mortem on a completely collapsed structure. Although there is a wide variety of bridge details used in California and in other countries, there is a consistency in the seismic damage experienced.

Even though bridges can be retrofitted to increase their resistance to total collapse in the event of a major earthquake, they will still experience minor damage from smaller seismic events.

Most of the retrofitting done to date has consisted of tying units of the superstructure together and to their supports. Although this directly solves only the problem of the spans dropping off of their supports, it partially alleviates some of the other seismic deficiencies.

The California bridge most seriously damaged since the 1971 earthquake would have sustained relatively minor damage rather than losing two spans if it had been retrofitted with restrainers.

Retrofitting Philosophy

The goal of retrofitting is to increase the seismic resistance of a bridge to minimize the probability of total collapse.

Retrofitting should eliminate or reduce the hazard to human life as much as possible.

If practical, critical bridges should be able to carry emergency vehicles after being damaged.

It is not practical or economically feasible to retrofit a bridge so that it will have the same seismic resistance as a new structure designed to current specifications.

Retrofitting is generally not recommended if the only expected deformation is a small probable maximum vertical displacement ($\leqslant$ 6") and some traffic can be accommodated by ramping the vertical offset with dirt or other readily available material until permanent repairs can be made.

Main spans of Pedestrian Overcrossings that could drop on vehicular traffic should be retrofitted. Other spans need not be retrofitted unless they can be done at a low cost when the main spans are done or unless there is a considerable amount of school or other high volume pedestrian traffic that could be injured.

Considerations for Retrofitting

It is not possible to formulate simple rules to determine whether or not a structure requires retrofitting to improve its performance during an earthquake or, if so, what type of retrofitting it requires. In addition to the geological and seismological conditions at a particular site, any one of a combination of two or more of the following physical features of a bridge could determine whether or not retrofitting is advisable.

- Type of construction
- Physical condition of the bridge
- Length
- Width
- Ratio of length to width
- Skew
- Curvature
- Number and location of joints
- Type of bearings and hinges
- Abutment type and height
- Bent type and height
- Number of spans
- Restraining devices (shear blocks, curtain walls, etc.)
- Type and degree of failure anticipated if not retrofitted.
- Column reinforcement details
- Lifeline requirements
- Sociological considerations
- Utilities carried

The following figures illustrate some of the different conditions that should be considered in determining whether or not a bridge should be retrofitted and the type of retrofitting that is required. It should be remembered that adverse geological conditions may complicate many of these situations.

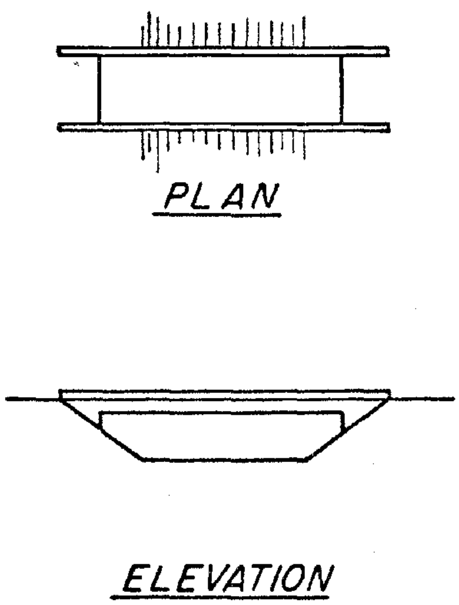


Figure 1

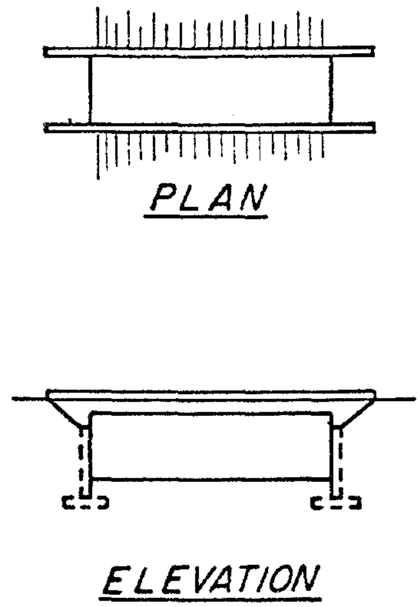


Figure 2

<u>Figures 1 & 2</u> As a rule, single span square structures should not require retrofitting. Although they may sustain some damage, they should be serviceable unless they cross a fault. If they do cross a fault, it is not likely that retrofitting will be effective.

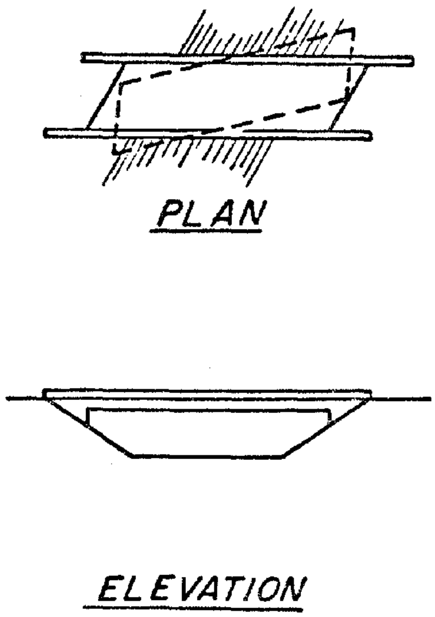


Figure 3

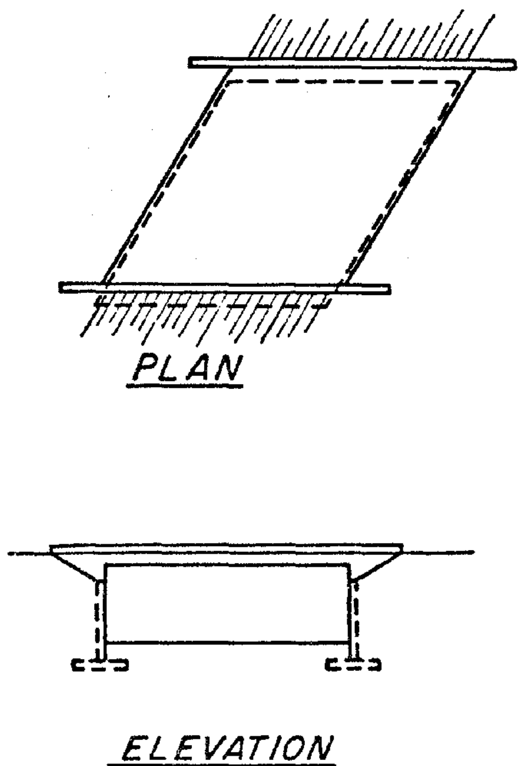


Figure 4

<u>Figure 3</u> Skewed bridge spans have a natural tendency to rotate even when not shaken. Longitudinal seismic shaking produces transverse components of force which tend to rotate the span each time it moves back and forth. Transverse seismic forces cause one end of the span to bear against one abutment while the opposite end tends to swing free -- in the natural direction of rotation. If the bearings, curtain walls or other means of restraining rotation fail, the span can rotate excessively. In some cases the span may drop only a few inches and the bridge can be used with minor inconvenience and easily restored to its pre-seismic condition. If the supporting seats are very narrow, the span can drop and the bridge will be a total loss.

<u>Figure 4</u> If a bridge is very wide in relation to its length, it may be locked between its abutments so that the rotation described in Figure 3 is negligible. Longitudinal shaking may cause insignificant damage. Transverse shaking may damage the bearings, shear keys or curtain walls, but there is much less probability for the more serious damage that might be expected with a longer, narrower structure.

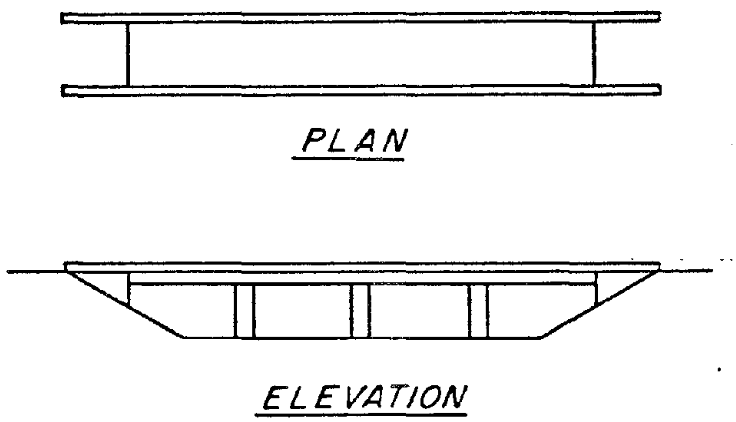


Figure 5

<u>Figure 5</u> Long, non-skewed, continuous bridges with diaphragm type abutments and without intermediate hinges or joints need not be retrofitted. Bridges with bearings at the abutments may require transverse restrainers at the abutments if it is determined that there is insufficient restraint provided by bearings, curtain walls, shear keys or other restraining features.

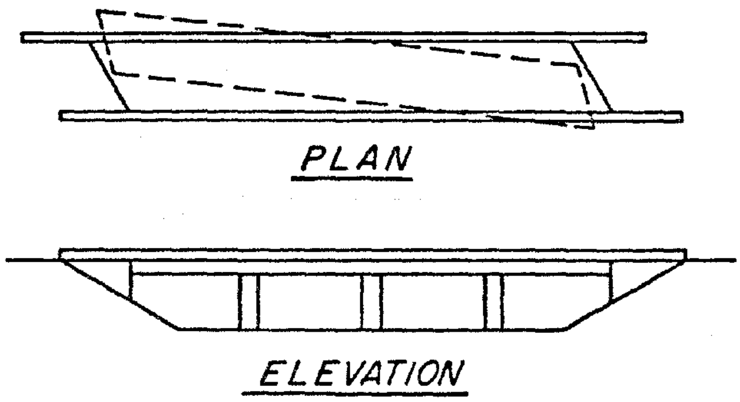


Figure 6

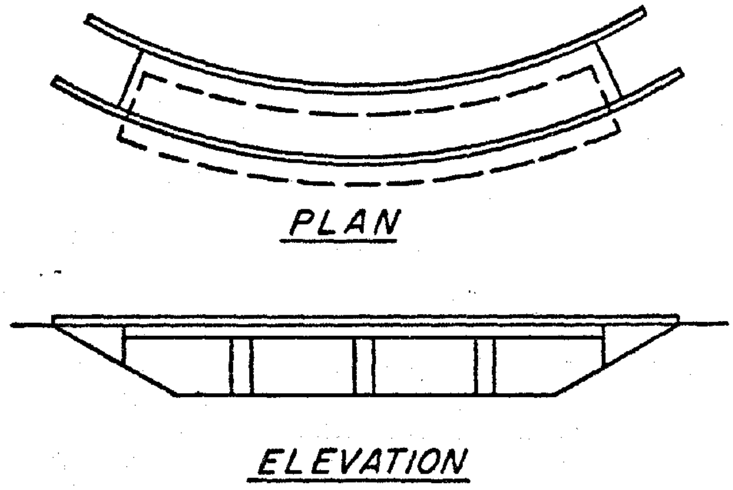


Figure 7

<u>Figures 6 & 7</u> Long, continuous, skewed or curved bridges without intermediate hinges or joints are more prone to seismic damage than similar square bridges. Due to the nature of the details, they will probably require additional transverse restraint at the abutments for a lower seismic level of shaking than a similar square bridge.

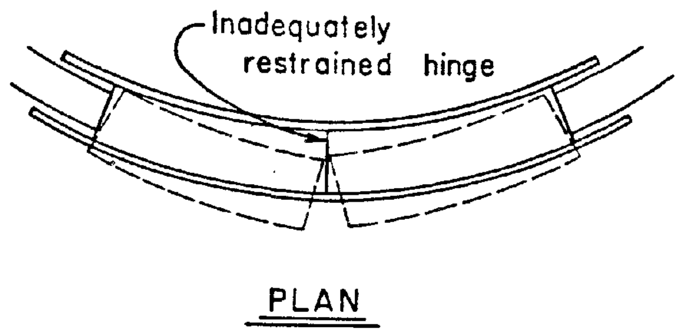


Figure 8

Figure 8 Segments of a superstructure which aren't adequately restrained act independently and may tend to fly apart when shaken. If the bearings or other means of transverse restraint fail, longitudinal restrainers (if installed) may act as tension members i a large horizontal beam. Restrainers should generally be placed as close to the edge of the structure as possible so they can offer the maximum amount of resistance for this condition.

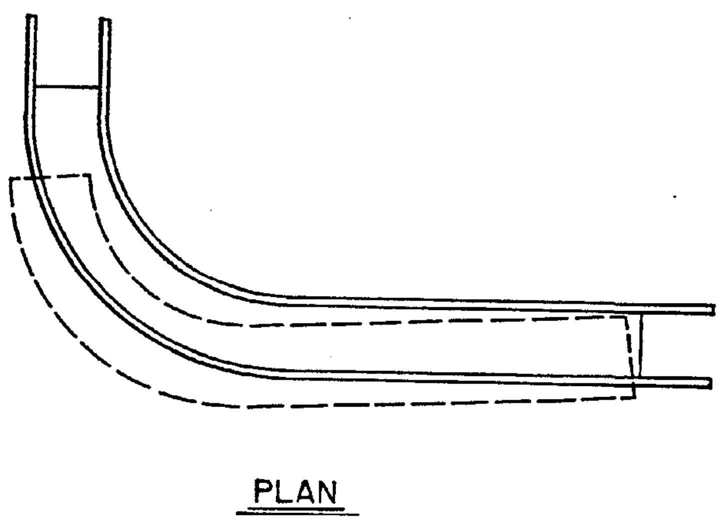


Figure 9

Figure 9 Sharply curved bridges which have seismically inadequate bearings at an abutment and very flexible or seismically deficient columns may require additional restraint at the abutments. Abutment restraint, in cases such as this, may alleviate some column weaknesses. One common problem, however, is that the abutments may not be capable of resisting the anticipated forces. Many abutments are very lightweight and the shear resistance of the soil or piles may be insufficient

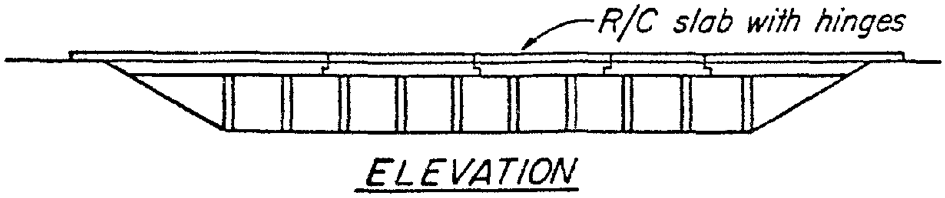


Figure 10

Figure 10 Long continuous reinforced concrete slab bridges, as a general rule, need not be retrofitted with hinge restrainers. This is based on the assumption that if the suspended span becomes unseated, the dead load of the resulting cantilever will not be sufficient to make it fail. Long span non-standard slabs should be checked for this criteria. Retrofitting should be required if there are two hinges in the same span or if the unseating of any hinge will lead to a dropping of any or all spans with dead load only. It is assumed that those unsupported ends will be quickly recognized and reseated, temporarily strutted, or traffic barricaded from using the bridge before any serious accident occurs.

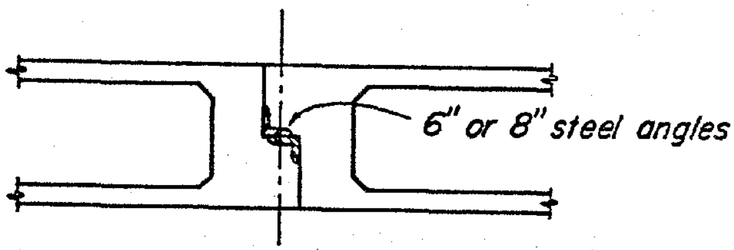


Figure 11

Figure 11 Any bridge with 6" or 8" steel angle hinges, or equivalent, should be retrofitted regardless of what seismic area it is in. Due to shrinkage, seasonal varations, and other factors, many of these hinges have marginal seating length under even normal conditions. Any seismic shaking could cause them to become unseated.

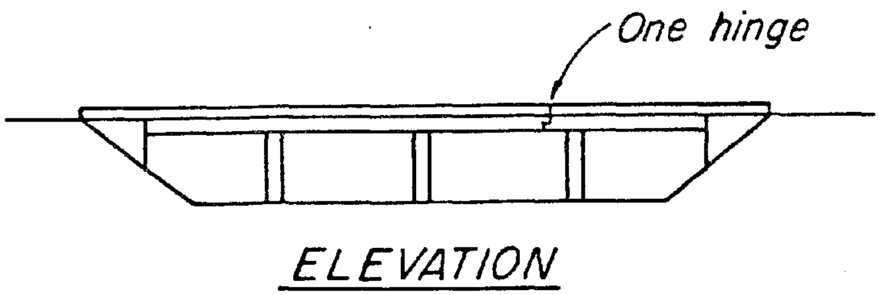


Figure 12

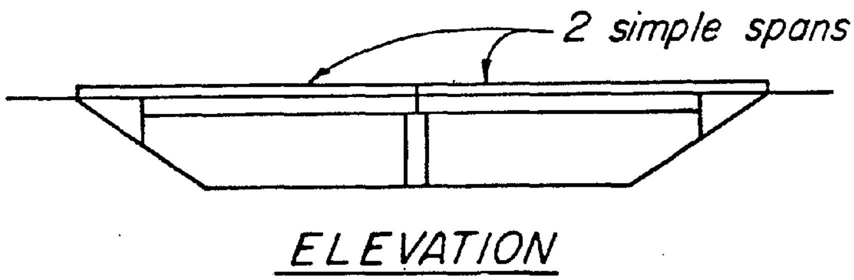


Figure 13

Figures 12 & 13 A non-skewed, straight, continuous bridge with only one hinge or a non-skewed straight bridge with two simple spans may be designed for the minimum of 25% (33% for LFD) of the dead load of the lighter segment of superstructure connected. This would be consistent with the rough assumptions made for the resistance and action of the earth behind the abutments.

The influence of the earth behind abutments becomes relatively less important if the superstructure is curved or skewed. The equivalent static force method or dynamic analysis should be used for designing restrainers for these structures if they are skewed or curved.

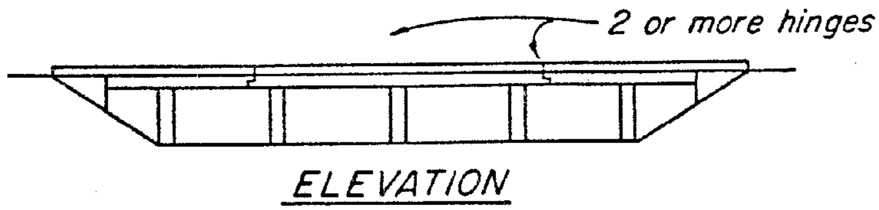


Figure 14

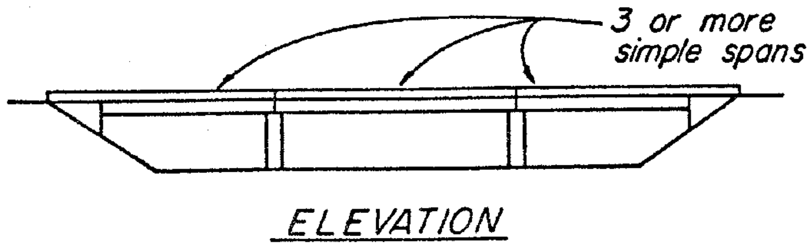


Figure 15

Figure 14 & 15 A dynamic analysis should be made for any bridge with two or more hinges or three or more simple spans.

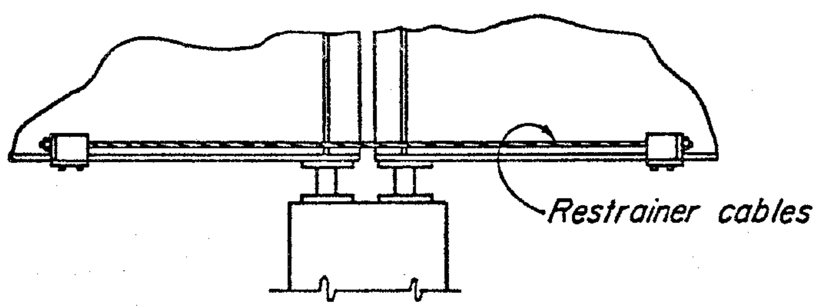


Figure 16

Figure 16 Connecting the ends of girders together in adjacent spans may be satisfactory for short structures with only a few spans and wide bent caps where it seems certain that the ends of girders won't drop off the bents. This detail can also be used where it is considered that the additional longitudinal forces produced by connecting the girders to the bent caps (see Figure 17) may fail the columns. Although the bearings will probably fail, the superstructure will not fall very far and the bridge will not be completely out of service.

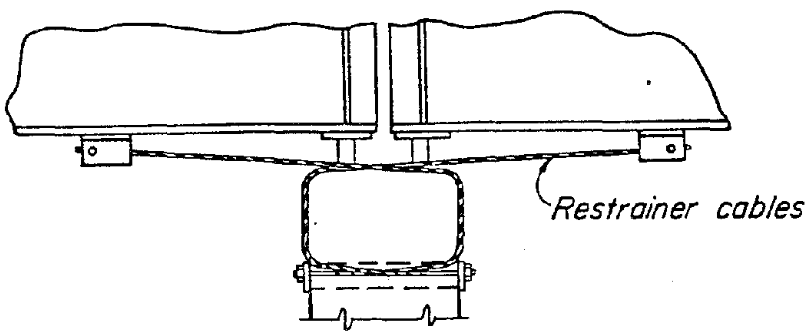


Figure 17

Figure 17 This detail is generally preferred to the one illustrated in Figure 16 with the spans butting against each other. The restrainers must be able to resist the force produced by both spans supported on that pier and possibly adjacent spans as well. Vertical clearances under the structure should be considered.

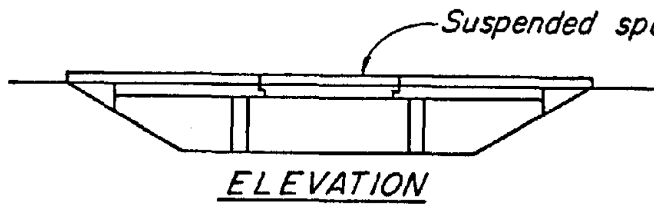


Figure 18

Figure 18 Suspended spans are particularily vulnerable to seismic shaking. Curved and skewed alignments greatly increase their vulnerability.

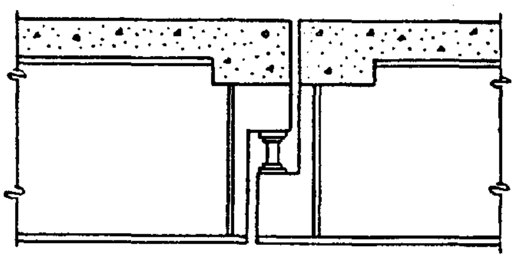


Figure 19

Figure 19 It can generally be assumed that any seat type hinge used with steel girders will need additional transverse, longitudinal, and vertical restraint in even moderately severe seismic areas.

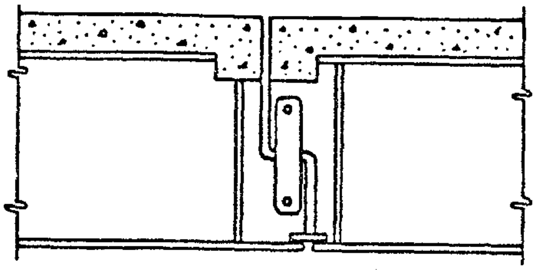


Figure 20

Figure 20 Hanger type hinges generally have more seismic resistance than the seat type shown in Figure 19, but are still subject to seismic damage. These hinges ofen have steel bars or angles that bear against the opposite web, or lugs attached to the flanges, which were designed to keep the girders aligned transversely for wind forces. Those devises are usually structurally inadequate and are too short to be effective with even moderate seismic shaking. Consideration should be given to replacing them or adding supplemental transverse restrainers.

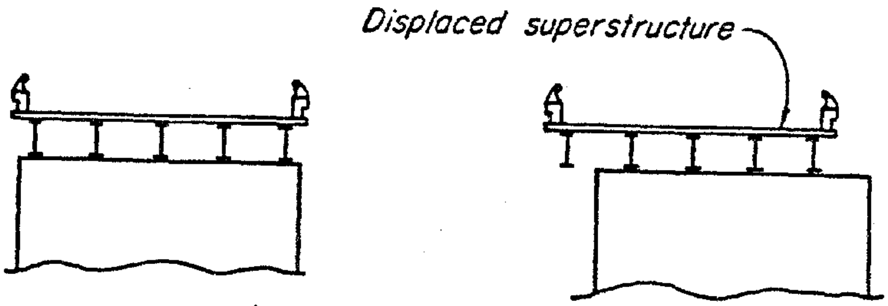


Figure 21

Figure 21 Very few older bridges have bearings that will not fail in a moderate or greater earthquake. It should be anticipated that a bridge superstructure can be displaced transversely. If the exterior girder of a multi-girder bridge is moved beyond the end of a bent, it is likely that that side of the bridge may be severely damaged and the use of a shoulder or lane will be lost, but traffic can be routed over a portion of the bridge with few or no emergency repairs. This is considered to be an acceptable risk.

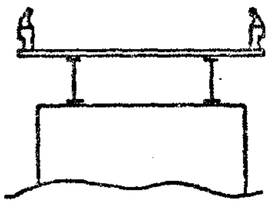


Figure 22

Figure 22 If the superstructure of a two or three girder bridge is displaced transversely so that one line of girders loses its support, the entire bridge may collapse. Adequate transverse restraint should be provided.

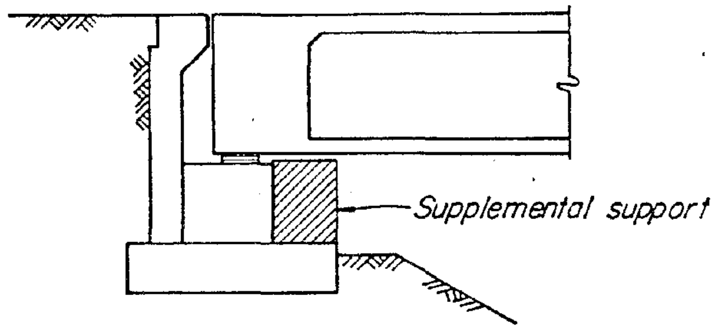


Figure 23

Figure 23 In most locations it is generally not practical to restrain longitudinally the superstructure at an abutment. Supplemental supports can be provided to prevent the superstructure from dropping excessively. This same principle may also be applied at bents in certain circumstances.

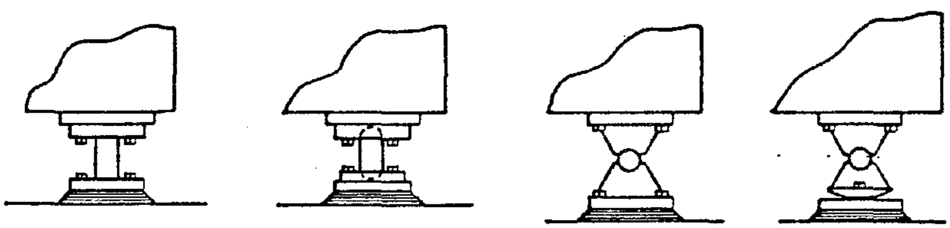


Figure 24

Figure 24 Numerous types of steel bearings used on various types of steel and concrete bridges have been damaged by relatively minor seismic shaking. It should be assumed that they will fail in areas where the maximum credible bedrock acceleration is 0.3g or greater. If the failure of any type of bearing will result in the superstructure dropping 6" or more without falling off of the pier or abutment, consider replacing the bearings with a modern type or adding bolsters that will minimize the drop.

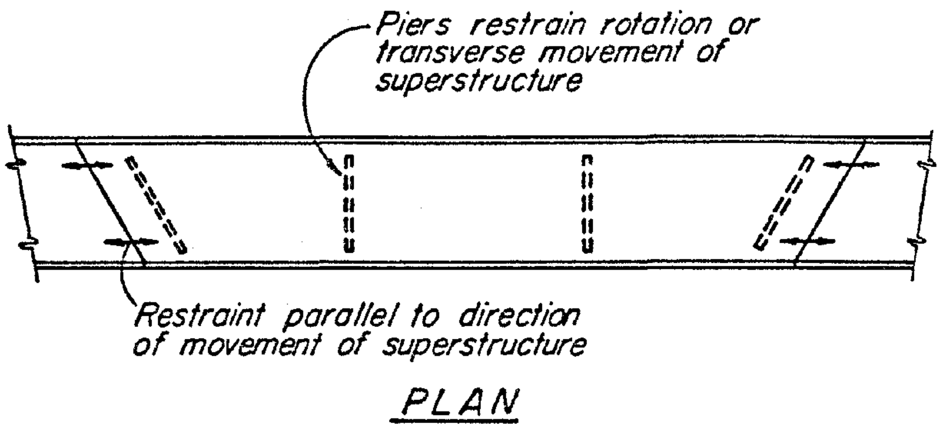


Figure 25

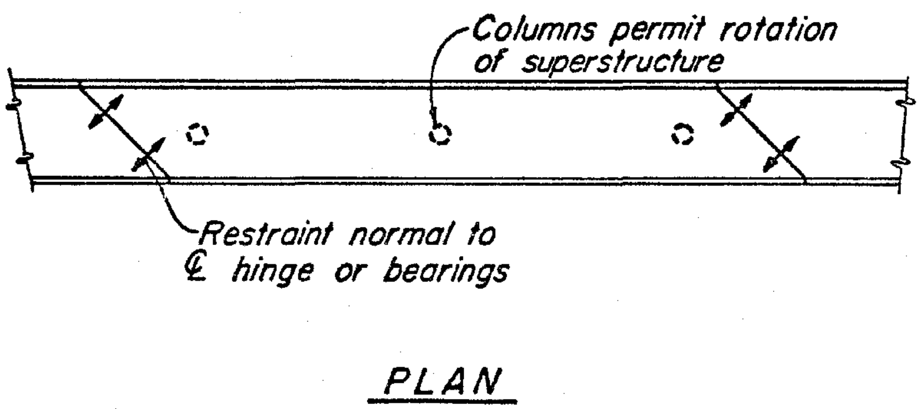


Figure 26

Figures 25 & 26 The rigidity of piers and bents can control the direction of movement of a structure. Restrainers will be more effective if they are oriented in the principal direction of movement.

Restrainer Requirements

Hinge and bearing restrainers should have redundancy. There is always a chance that a single unit has a defect (due to faulty material, fabrication, installation, adjustment, maintenance, etc.) and will fail sooner than expected. Additional units or other devices should be capable of doing their share of the job if one unit fails prematurely.

Restrainers should fail in a ductile rather than brittle manner when subjected to ultimate loading. They should not fail before the structure as a whole fails.

Restrainer brackets and connections should be at least 25% stronger than the cables, rods or primary restraining devices. They should be designed so that they will not fail or cause failure of the portion of structure they are attached to if some component part or parts of the unit are misadjusted or fail prematurely.

The following ultimate strengths should be assumed for designing connections and determining the adequacy of supporting members:

3/4" cables (6x19. Federal Spec. RR-w-410c)

$F_u = 53$ kips

1¼" H.S. rods (ASTM A-722 with Supplemental Requirements)

$F_u = 188$ kips

(use 53 x 1.25 = 66.2 kips and 188 x 1.25 = 235.0 kips per cable and rod, respectively)

Bolted Connections shall be designed as a bearing type:

H.S. Bolts (A325)	Allowable Shear ($F_v = 0.6\ F_u\ \phi\ A_r$)	Allowable Tension ($F_t = \phi\ F_u$)
3/4"	20.1 kips	34.1 kips
7/8"	27.7	47.1
1"	36.3	61.8
1 1/8"	45.8	68.1

Combined Tension and Shear:

$$F_{vc} = \sqrt{(F_v)^2 - (0.6\ f_t)^2}$$

Where: F_{vc} = Allowable shear per bolt for combined shear and tension

ϕ = Reduction Factor = 0.85

F_v = Allowable shear per bolt (kips)

f_t = Applied tension per bolt (kips)

A_r = Area of bolt with threads in shear plane

F_y = Ultimate tensile strength (kips)

The following allowable stresses should be used for designing ASTM A-36 steel brackets for ultimate conditions:

Tens. or Comp. = 36,000 psi
Shear = 22,000 psi

Bearing $\frac{L F_u}{1.18 d}$ or $3.0 F_u$ whichever is smaller

F_u = 58,000 psi

L = Distance in inches measured in the line of force from the centerline of bolt to the nearest edge of the hole for an adjacent bolt or to the end of the connected part toward which the force is directed.

d = Diameter of bolt in inches.

F_u = The lowest specified minimum tensile strength of the connected parts.

L/d shall not be less than 1.5

Groove welds = 36,000 psi
Fillet welds = 26,000 psi

Bearings:

One of the primary seismic weaknesses of older bridges is that the spans are not connected to each other or to the bents and abutments. Bearings usually provide the only connection between these units. Experience has shown that the seismic resistance of bearings is often overrated and they are damaged by relatively minor shaking.

When seismic shaking becomes more severe or prolonged, damaged bearings offer no restraint and allow the spans to fall off their supports.

As a general rule, a designer should be very cautious about assuming that bridge bearing anchor bolts, keeper bar bolts or welds and similar details have any significant effect in keeping a bridge superstructure on its supports during a major earthquake. The following shortcomings of bridge bearings should be considered:

1. All of the bearings at the end of a span probably are not subjected to identical forces simultaneously. Because keepers or other devices are not set with exactly the same clearances, only one half, or fewer than one half, of the bearings will initially resist a horizontal force in one direction. Rotation of a span in a horizontal plane puts unequal loads on the bearings. It is not uncommon for bearings at one end of a span to be damaged to varying degrees by an earthquake.

2. Grout pads under bearing masonry plates have traditionally given trouble during and after construction and have been one of the main sources of trouble in minor quakes. Failure of a grout pad will allow the bearing assembly to move and subject the anchor bolts to combined bending and shear.

3. The common detail of a girder seated on an elastomeric pad will subject anchor bolts to combined bending and shear.

4. Anchor bolts are frequently threaded below the top surface of the pier or abutment seat. This gives a reduced area for shear and minimal resistance to bending before failure occurs due to notch sensitiveness at the root of thread.

5. Although it is less common, some anchor bolts are too close to the edge of the bearing seat, have inadequate reinforcement around the bolts, and will spall off the concrete when subjected to horizontal loads.

6. Keeper bars allow movement between the sole plate and bearing bar or rocker. Sliding takes place on this surface. Sliding obviously does not start until the horizontal force exceeds the vertical load times the coefficient of friction. When this happens, does it result in an impact on the keeper bar and anchor bolts? If so, it can increase the calculated force considerably.

7. Bridge bearings may not be what they are represented to be on "As Built" plans or maintenance records. Adjustments to keepers or other details are occasionally made after construction is completed and the details or workmanship may be inferior to the original.

Earthquake restrainers should be considered if the strength of the bearings is less than twice the calculated seismic force on them, after taking due consideration of the above deficiencies and uncertainities.

Columns

Many older bridges have seismically deficient columns as well as inadequate bearings. The deficiencies may be due to an insufficient amount of longitudinal reinforcement; too few, too small or improperly detailed ties or spirals; improperly located lap splices; or inadequate anchorage or confinement of longitudinal steel in footing or caps. These deficiencies are much more critical for structures with single columns than those with multi-column bents. The problem is so extensive and costly to correct that most structures with seismically deficient columns (especially those with multi-column bents) will never have those deficiencies corrected. In some of those cases the calculated forces required for hinge restrainers may be greater than the columns can resist. If it is obvious that deficient columns will not be retrofitted, consideration should be given to limiting bearing restrainer forces to approximately 25 percent greater than what is required to fail the columns.

DEVELOPMENT OF POST-EARTHQUAKE MEASURES FOR BUILDINGS AND STRUCTURES DAMAGED BY EARTHQUAKES

by

E. Kuribayashi
M. Hirosawa
T. Murota

PURPOSE OF THE PROJECT

Japan which locates in the Circum Pacific Seismic Belt is one of the famous earthquake-hazardous countries in the world and suffered from major seismic disasters many times in history. Those disaster experiences have urged the development of aseismic engineering for buildings and structures.

The dramatic development in recent years has enabled us to construct gigantic structures such as long-span bridges and high-rise buildings.

In addition, a research project "Development of New Aseismic Design Methods for Buildings and Structures" performed by the Ministry of Construction from 1972 to 1976 based on the lessons obtained from Tokachi-Oki earthquake in 1968 and San Fernando earthquake in 1971, has led the aseismic design method for buildings and structures to a higher level.

As a result, major damages to buildings and structures such as total collapse have decreased and correspondingly human life has become to be kept in safe. However, those minor damages or partial failures as observed in recent earthquakes, i.e., Izu-Oshima earthquake or Miyagi-Oki earthquake, have yet been expected to occur in future earthquakes. The measure to be taken for those buildings and structures is a matter of significant concern to government jurisdictions.

In El-Asnam, Algeria earthquake and South Italy earthquake occurred continually on October and November of 1980, there were observed the aftershock damage to buildings the resistance of which had been weakened by the initial shock and also observed the magnification of disaster caused by the delay of urgent helps or post-earthquake inspection and repair. These facts suggest the importance of appropriate post-earthquake measures.

Based on the background, the purpose of this project is to present post-earthquake measures by developing inspection methods, assessment methods and repair and strengthening methods for buildings and structures damaged by earthquake.

OBJECTIVES

This project starts in 1981 fiscal year and scheduled to end in five years. The objectives are as follows:

1. Development of inspection methods.

A. Measurement of structural performance.

The objective of this section is to present a manual for field measurement of residual structural performance of buildings and structures subjected to earthquake damage by utilizating the preceding technics.

B. Inspection methods

The objective of this section is to provide guidelines for the earthquake resistance inspection to make an immediate and appropriate decision to repair, strengthen, demolish a building or place off-limits.

2. Development of repair and strengthening methods

The objectives of this chapter are listed below:

a. To provide criteria for aseismic performance levels concerning rigidity, strength, ductility, etc. which have to be provided by buildings and structures to be used or occupied after the earthquake.

b. To develop the repair and strengthening methods suitable for various modes and extents of failure including expreimental examination.

c. To present guidelines for repair and stengthening.

d. To perform experimental examination on the earthquake earth pressure in slopes, fill grounds and retaining walls corresponding to soil and topographical conditions.

e. To provide guidelines for repair and strengthening of slopes, fill grounds and retaining walls based on the experimental examination on the effectiveness against future earthquakes and rains.

3. Development of assessment methods

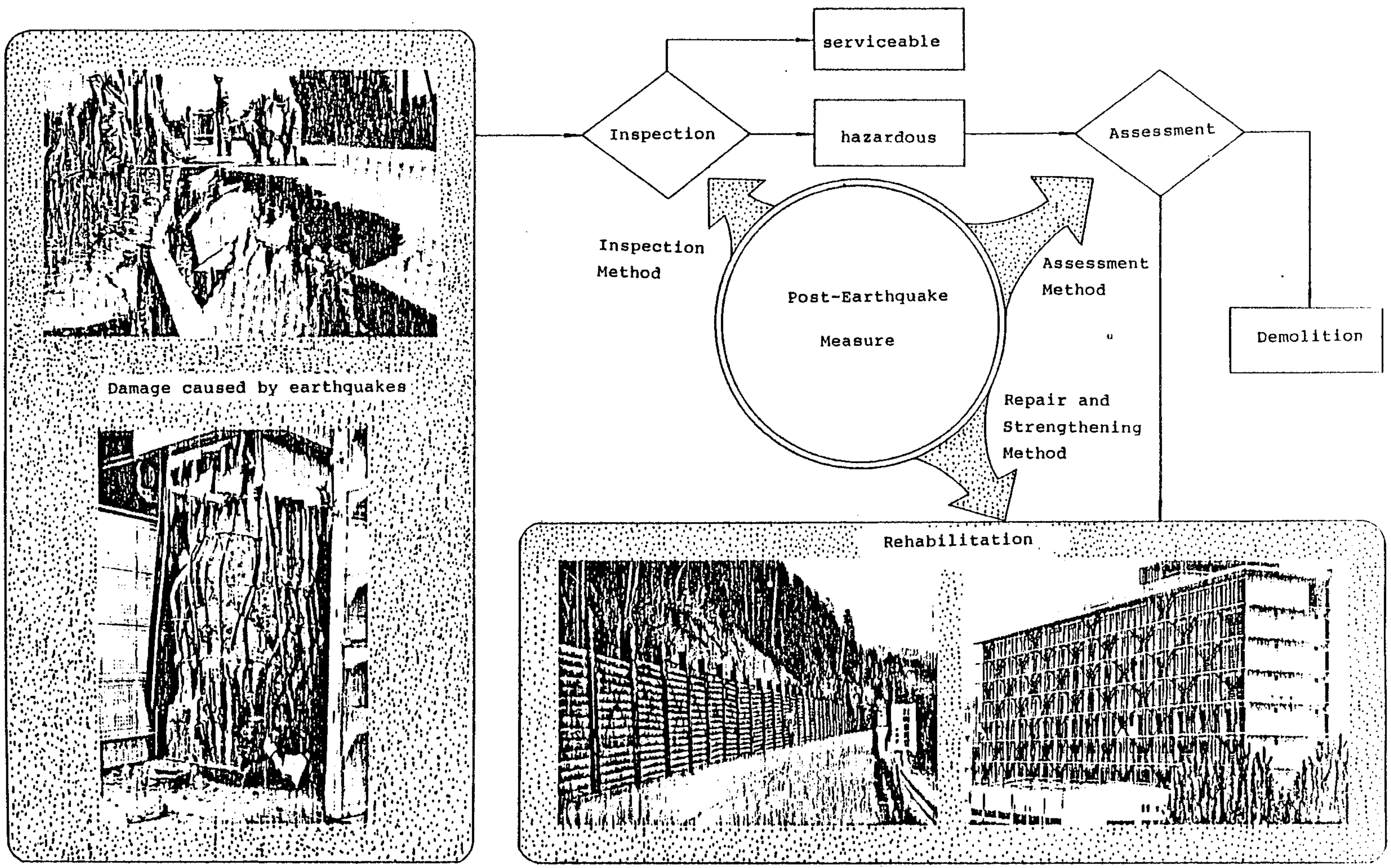
A. Factors in assessment

The objective of this section is to list up and analyse the factors in considering a repair or strengthening program such as aseismic performance to be provided, period and workability of repair works, functional damage during the period, spectacles imparied by repair works, repair costs and other economic benefits.

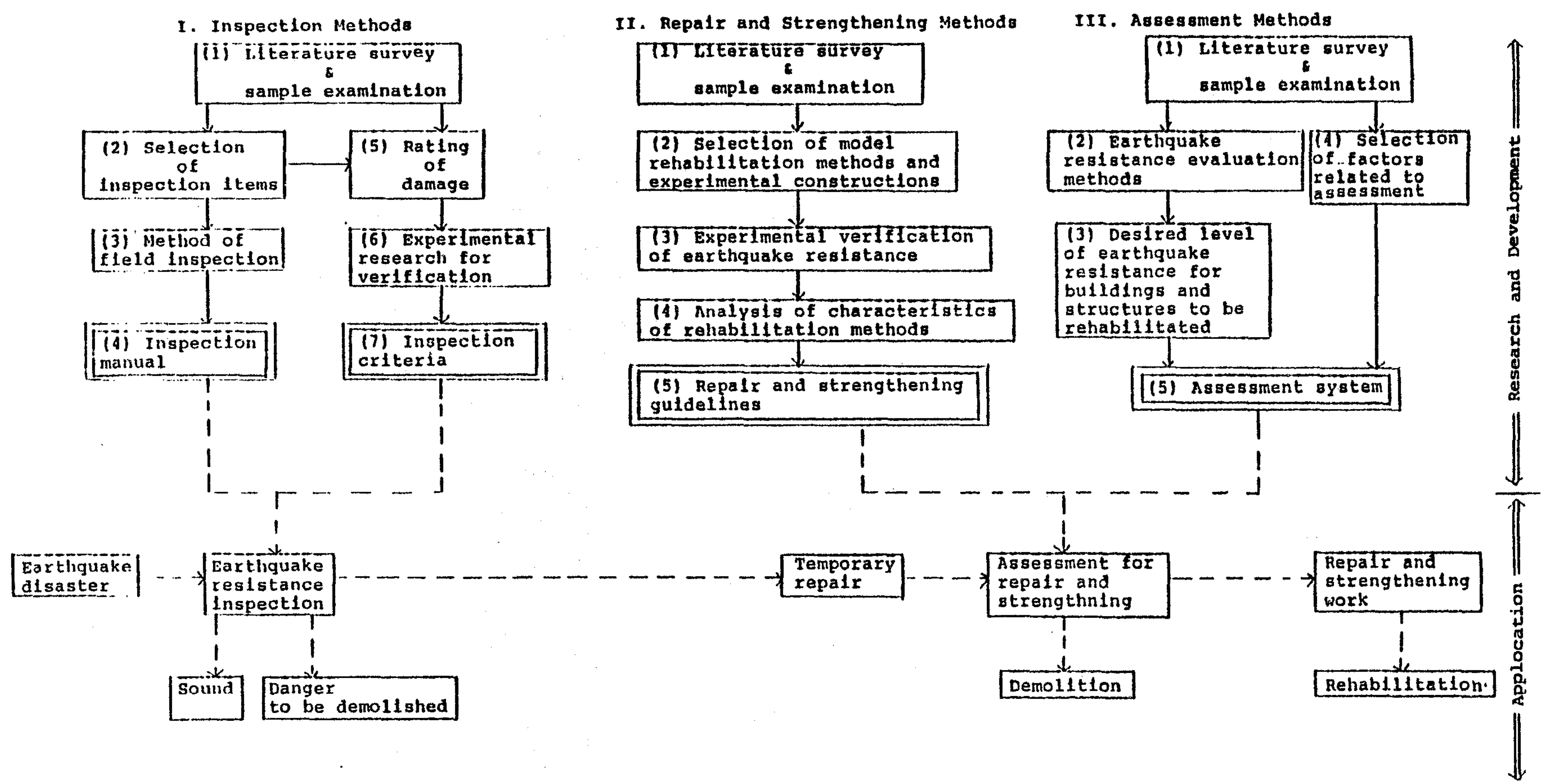
B. Methods of assessment

The objective of this section is to provide the methods for evaluation of earthquake resistance restored by repair and/or strengthening and also to develop a guide for determining appropriate repair and strengthening methods based on the consideration of factors referred in 3A.

Development of Post-Earthquake Measures for Buildings and Structures Damaged by Earthquakes



Flow Chart of the Project



CRITERION ON THE EVALUATION OF SEISMIC SAFETY OF EXISTING REINFORCED CONCRETE BUILDINGS

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Building Research Institute
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CONTENTS

1. Introduction

This report describes the outline of "Criterion on the Evaluation of Seismic Safety of Existing Reinforced Concrete Buildings" which was compiled by the joint committee chaired by Dr. H. Umemura, Prof. of Tokyo University, with commition from Ministry of Construction, Japanese Government.

The buildings which this criterion covers are low-and medium-rise reinforced concrete buildings by ordinary construction method and items for evaluation are not only super structure itself but non-structural elements such as exterior finish elements. Further, these evaluation methodologies are consisted of three steps from the simple first screening to the complicated third screening.

The result of evaluation is expressed by the continuous numerical value but the result shall be judged by the engineer who uses this criterion considering individual and social impact caused by the presumed damage.

Moreover, the applied results on damaged and un-damaged buildings in the Tokachi-Oki Earthquake, 1968 are shown as the refference for the judgement.

In the following, several features and the whole text of the criterion are described.

2. Several Features of Criterion

2.1 Adoption of Seismic Index of Non-structural Elements

The seismic safety of buildings should be examined not only from a viewpoint of the safety of structural elements from collapse, but also from the viewpoint of the safety of non-structural elements such as finishing materials of exterior walls directly facing to streets from their fall. Because that reinforced concrete buildings in Japan have relatively large lateral strength, the structures, itself seemed to seldom fall down instantaneously even under the strong earthquake motions. Actually, in the experience of past earthquake damages, most of buildings survived from the catastrophic destruction. Even in cases of buildings which were unfortunately destroyed and fell down, the residents of the buildings had enough time to escape from the buildings.

Therefore, it becomes important to protect people from injury of the fall of the non-structural elements such as finishing materials of exterior walls.

Though there is no sufficient experimental and empirical information concerning about the performance of non-structural elements under earthquake loads, the safety evaluation of non-structural elements by I_N-index is attempted in this criterion taking into account of the relative flexibility of structure itself and non-structural elements.

2.2 Adoption of Screening Method

The structural safety evaluation considered in this criterion consists of a sequence of steps from 1st to 3rd evaluation. This procedure is repeated in successive cycles, the assumptions and details of the calculations being refined in each successive cycle when necessary for a reliable estimate of structural performance. The repetitive procedure is called "Screening", and is believed to be the fastest and the most practical method for reasonably evaluating the structural adequacy of a large number of buildings subjected to strong earthquake motions.

2.3 Evaluation and Judgement of Seismic Safety

The evaluation of the seismic safety in a broad sense is taken more precisely in the two following senses:

1) Evaluation of seismic safety; to express the seismic safety of

structures with continuous quantity such as seismic index proposed in this criterion.

2) Judgement of sesimic safety; to judge the adequacy of buildings for seismic safety taking account of various conditions such as their use, their importance and their age, based on the seismic index obtained by the evaluation of seismic safety.

This criterion aims to evaluate the seismic safety as defined above and the judgement is left to the engineers who use this criterion.

The applied results of this criterion on damaged and un-damaged buildings in Tokachi-Oki Earthquake of 1968 are summaized in the appendix of this criterion. These results will be helpful in performing the judgement of the seismic safety.

2.4 Adoption of Seismic Sub-indexes, S_D, T, and G to Seismic Index, I_S

Seismic sub-indexes, S_D and T which represent the quality of structural design and time dependent deterioration respectively, are taken into account in this criterion as the sub-indexes of synthesis index representing seismic safety, I_S in addition to the seismic sub-index of basic structural performance E_o which related to the lateral load carrying capacity and the deformation capacity of structures. In this criterion, the quantitative evaluation of such sub-indexes are attempted using check list system. Moreover, seismic sub-index G representing the intensity of input ground motions to the base of a building, which depends on the seismicity of its location an on the relationship between its dynamic characteristics and the kind of soil is defined as G in this criterion. The standard value of this sub-index is taken as equal to 1.0 and decreasing value with increase of the earthquake danger in the location is assumed. However, G-index is fixed to 1.0 in this criterion because of the difficulty of the evaluation of the earthquake danger at present.

2.5 Consideration of Seismic Sub-index of Ductility, F to Seismic Sub-index of Basic Structural Performance, E_o.

In the basic sub-index representing the earthquake resistant ability of structures, E_o, not only strength but also deformation capacity are considered as follows.

1) Critical conditions defined by the failure of brittle members.

The lateral load carrying capacity of a building depends on the failure of brittle structural members provided that the building is consisted of structural members with various deformation capacity and, therefore, is not always the sum of the ultimate lateral strength of every structural members. In general, critical displacements at the ultimate strength of brittle structural members are small because of their high stiffness, and then the ductile members which have relatively low stiffness might not reach their ultimate strength at the critical displacements.

Moreover the brittle members show significant reduction of load carrying capacity after they reached their ultimate strength.

Therefore, the failure of brittle structural members becomes one of critical conditions for evaluating the seismic performance of buildings. In this criterion, such critical conditions are expressed in Eq. (2), (3) and (5). In these equations, α means one of the reduction factor of the strength for ductile members considering the compatibility of the displacement at the failure of brittle members. The value of α which is taken as 0.5 to 0.7 in these equation is determined empirically, based on many test results on the yield displacements.

On the other hand, the failure of brittle members causes often the local collapse of buildings because that they become ineffective to sustain vertical loads. Therefore, in this criterion, the failure of brittle members is considered to be one of the critical conditions on the safety of buildings even if the lateral load carrying capacity of the buildings as a whole is not affected by it. Such a critical condition is considered in Eq. (3) or (5).

2) Critical condition of buildings consisted of the structural members which have various deformation capacity.

It is not always easy to evaluate the seismic safety of the buildings consisted of the structural members which have various deformation capacities. In case of a building consisted of structural members which have almost same deformation capacity, it is possible to evaluate its earthquake resistant, based on the assumption of the equal energy concept proposed by Blume et al, which implies that the potential energy stored by the elastic system at maximum deflection is the same as that stored by the elastoplastic system at maximum deflection. In case of a building consisted of, for example, some brittle shear walls and ductile columns, its seismic resistant ability changes with change of the ratio of the load carrying capacity of walls to that of columns or change of deformation capacity of framing members. For evaluating the seismic safety of such type of structures, Eq.(4) is proposed, based on the many non-linear dynamic analyses of combined structures of brittle shear walls and ductile frame responding to ground motions recorded during severe earthquake.

3) Relation between required ductility factor of non-linear system, and seismic sub-index, F.

Non-linear dynamic analyses of structures responding to earthquake motions have shown that the required ductility factor of the elasto-plastic systems whose yield shear factor, is C_y may be estimated from the elastic spectral response acceleration, C_E. Blume et al, for example, has shown that the required ductility factor of reinforced concrete structures is given by the following equation;

$$C_E/C_y = \sqrt{2\mu - 1}$$

where C_y ; yield shear factor of elasto-plastic system.

C_E ; spectral response acceleration of elastic system.

μ ; required ductility factor of elasto-plastic system.

This equation is based on the equal energy concept as mensioned in the Article 2). Comparing the above equation with results obtained from dynamic analyses on single degree of freedom systems with elasto-plastic and degrading stiffness load-deflection relationship, it is evident that the above equation may be an upper bound.

For determining the seismic sub-index, F given in Eq.20, the same approach as mensioned above has been applied, based on the nonlinear dynamic analyses responding to the gound motions recorded during severe earthquake carried out on the single degree of freedom oscillator having degrading tri-linear load-deflection relationship which seemed to be a typical load-deflection relationship of reinforced concrete structures.

The reciprocal of the seismic sub-index, 1/F in this criterion is one of the upper bound of the ratio of the yield shear factor of degrading tri-linear system to the elastic spectral response acceleration.

4) Determining the required ductility factor of structural members of multistory frames from the reponse ductility factor obtained from non-linear dynamic analyses of one mass system.

The ductility demand obtained from the non-linear dynamic response analyses on the one mass system cannot be claimed to give an accurate assessment of the ductility demand of each structural members of the multistory frame responding to non-linearly to strong earthquakes. In this criterion, however, it is supposed that the ductility demand of each structural members assumed to be the same as the response ductility factor obtained by the non-linear dynamic analyses of one mass system.

Many experimental studies have been carried out recently on the ductility behaviour of the flexural yield type structural members. However, there is a lack of information concerning about the quantitative estimation of allowable ductility in accordance with structural details of the members. The equation (22) is proposed provisionally for estimation of the allowable ductility of flexural columns with some restrict conditions in which ductil behaviour can not be expected.

In cases of walls, even the experimental studies on the ductility behaviour have not been performed sufficiently. Therefore, the F-index is directly given by Eq. 24 for walls for safe side estimation instead of the estimation of F-index from the allowable ductility factor as in the case of columns.

2.6 Recommendation for repairs to improve the earthquake resistant characteristics of buildings

When insufficient seismic safety of buildings comes into question as the results of the application of this criterion, appropriate repairs may be required for improving the earthquake resistant characteristics of the buildings. The recommendation for repairs are also provided for this purpose. This recommendation deal with the procedures of repairs in accordance with strength requirements or ductility requirements of the structural members. The method of the evaluation of the seismic safety of the repaired buildings, some attention for the practice of repairs, and some design details for repairs are also provided in this recommendation.

CONTENTS

1. General Rule

1.1 Basic Plan and Scope

This criterion is applied to low or medium-rise existing reinforced concrete buildings (they are called RC buildings for short hereafter) in the case of evaluating the seismic performance of them briefly, and it is composed of three evaluation methods. Each method has a different level from one anther, and is respectively named for the first evaluation method, the second evaluation method, and the third evaluation method.

In addition, this seismic evaluation is an expression of seismic capacity of a building by the continuous index. The decision on the result shall be performed according to the judgement standard that is established elsewhere.

1.2 Preliminary Investigation

Before applying this criterion, in accordance with a proper preliminary investigation, it is necessary to decide whether this criterion may be applied or not. The preliminary investigation is an approximate investigation about whether the structural plan, type and time-dependent condition of the building have much difference from those of normal buildings.

2. Definition of Seismic Index

The seismic safety of buildings is represented in the following two indexes, and the higher value of each index means the higher seismic safety.

I_S : Seismic Index of Structure

I_N : Seismic Index of Non-Structural Elements

They are in principle independent, however, their relations are a little considered. For instance, the relation between ductility of structure and ductility of non-structural elements is used for the culculation of I_N.

3. Calculation of Seismic Index of Structure, I_s

3.1 General

(1) Index of structure, I_s, is calculated by Eq.(1) about the longitudinal and ridge direction at each floor of a building. However, G-index, T-index and S_D-index in the first evaluation method are not related to the floor location and the direction.

$$I_S = E_0 \times G \times S_D \times T \qquad (1)$$

where, E_0 : Seismic Sub-Index of Basic Structural Performance (Section 3.2)

G : Seismic Sub-Index of Ground Motion (Section 3.3)

S_D : Seismic Sub-Index of Structural Profile (Structural Design) (Section 3.4)

T : Seismic Sub-Index of Time-Dependent Deterioration (Section 3.5)

(2) In calculating I_S-index, any one of three methods may be used (the first, the second and the third evaluation method). The generalization of each method is as follows. The larger the number of method is, the more detailed the calculation is and the higher the reliability is.

i) The First Evaluation Method

E_o-index is calculated by the ultimate strength that is approximately calculated from the ratio of wall and column sectional area to sum of floor area. S_D-index and T-index are calculated roughly on the same level with the calculation of E_0. This method is suitable for the building that has a lot of walls, and may underestimate the building that has few walls.

ii) The Second Evaluation Method

Based on the assumption that the strength of beams is sufficiently large, E_0-index is calculated by the ultimate strength of walls and columns (which is calculated by a little more detailed equations than those of the first evaluation method), failure mode, ductility and so on. S_D-index and T-index are a little more detailed than those of the first evaluation method, too. Because ductility together with strength is reflected in E_0-index, the value of E_0-index of the building, that have ductile framing structure, may be higher than the value calculated by the first evaluation method. Furthermore, the standard value for safety judgement in the case of the second evaluation method may be lower than the case of the first evaluation method as the reliability of the calculation by the former is higher than that of the calculation by the latter. The above mentioned matter is also true in the relation between the third evaluation method and the second.

iii) The Third Evaluation Method

When E_0-index is calculated, the type of yielding mechanism, the rotation of foundation under wall and etc. are taken under consideration. S_D-index and T-index are calculated in the same way as the second evaluation method. The seismic safety of buildings is investigated more minutely and the reliability of calculations is higher as compared with the second evaluation method.

3.2 Seismic Sub-Index of Basic Structural Performance, E_0

3.2.1 Calculation of E_0-Index

Based on the assumption that the other sub-indexes are 1.0, E_0-index shows the seismic performance of buildings by the ultimate strength, the type of failure mechanisms and the ductility. The larger the strength is and the higher the deformation ability is because of the ductile failure type, the higher the value of E_0-index is.

By combining strength index C, failure type (Section 3.2.2), ductility index F. (Section 3.2.3) and others, E_0-index at the i-th story of n-storied building is calculated as follows by each method.

(1) The first Evaluation Method

In the first evaluation method, vertical members of buildings are classified into three categories (Table 1), and E_0-index is calculated as follows.

Table 1. Classification of Vertical Members for the First Evaluation Method

Mame	Definition
column	independent column ($ho/D > 2$)
extremely short column	independent column ($ho/D \leqq 2$)
wall	including the wall not surrounded by framing members

notes : ho : clear height of column ; If there is upper or lower wall, ho becomes short. (Figure 1)

D : depth of column section

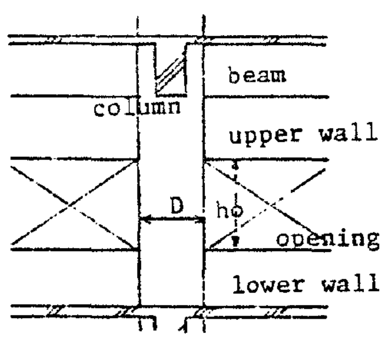


Figure 1. Clear Height of Column, ho

i) E_0-Index of Buildings without Extremely Short Columns

E_0-index is obtained by Eq.(2) in the case that there is no extremely short column.

$$E_0 = \frac{n+1}{n+i} (Cw + \alpha_1 \cdot Cc) \times Fw \qquad (2)$$

where, n : total number of stories of the building

i : the number of the story under investigation; 1 is used at the first story, and n is used at the top story.

Cw : strength index for walls by Eq. (7)

Cc : strength index for columns by Eq. (8)

α_1 : (sum of the lateral shear forces sustained by columns corresponding to the displacement at the ultimate strength of walls) / (sum of the ultimate strength of columns) ; 0.7 may be used for this value, however, it is 1.0 in the case of Cw = 0.

Fw : ductility index of walls (ductility index of columns in the case of Cw = 0) ; 1.0 may be used for this value.

ii) E_0-Index of Buildings with extremely short Columns

In the case that there are some extremly short columns, E_0-index is the higher value which is obtained by Eq. (3) or by Eq.(2) neglecting extremely short columns.

However, if the extremely short column is the secondary seismic element, Eq. (3) should be used. The secondary seismic element is the member which is permitted to fail by horizontal load and has no elements arround to support the vertical load that is sustained by the member at the failure.

$$E_0 = \frac{n+1}{n+i} (Csc + \alpha_2 \cdot Cw + \alpha_3 \cdot Cc) \times Fsc \qquad (3)$$

where, Csc : C-index of extremely short columns, calculated by Eq. (9)

Cw : C-index of walls, calculated by Eq. (7)

Cc : C-index of columns, calculated by Eq. (8)

α_2 : (sum of the lateral shear forces sustained by walls corresponding to the displacement at the ultimate strength of extremely short columns) / (sum of the ultimete strength of the walls) ; 0.7 may be used for this value.

α_3 : (sum of the lateral shear forces sustained by columns corresponding to the displacement at the ultimate strength of extremely short columns) / (sum of the ultimate strength of columns) ; 0.5 may be used for this value.

Fsc : ductility index of extremely short columns; 0.8 may be used for this value.

(2) The Second Evaluation Method

In the second evaluation method, first, we determine the failure type (Table 2) and ultimate shear force (3.2.2 (2), ii), iii)) of each vertical member at the objective story by the process shown in Section 3.2.2 (2), and we calculate the ductility index of each member by the process shown in Section 3.2.3. Next, we classify vertical members into three or less groups so that the members of which the failure tipes and ductility indexes are near each other are in one group, and then we calculate the structural indexes by Section 3.2.2 and the ductility indexes by Section 3.2.3 about the groups. Failure types are shown at Table 2. Vertical members classified into three or less groups are named for the first, the second and the third group according to the order from the lowest F-index. Lastly, E_0-index is calculated by combining the structural indexes C and ductility indexes F of each group as follows.

Table 2. Classification of Vertical Members by Failure Types for the Second Evaluation Method

failure type	Definition
flexural column	column that flexural yielding precedes shear failure
flexural wall	wall that flexural yielding precedes shear failure
shear column	column that shear failure precedes flexural yielding ; However, extremely brittle column is excluded.
shear wall	wall that shear failure precedes flexural yielding
extremely brittle column	column that ho/D is less than or equal to 2.0 (extremely short column), and shear failure precedes flexural yeilding

i) E_0-index of Building without Extremely Brittle Columns

In the case that there is no extremely column, E_0-index is the higher value which is calculated by Eq. (4) or by Eq. (5). However, if there are some shear columns which are the secondary seismic elements, Eq. (5) should be used.

$$E_0 = \frac{n + 1}{n + i}\sqrt{E_1^2 + E_2^2 + E_3^2} \tag{4}$$

where, E_1 : $C_1 \times F_1$

E_2 : $C_2 \times F_2$

E_3 : $C_3 \times F_3$

C_1 : C-index of the first group (F-index is lowest)

C_2 : C-index of the second group (F-index is middle)

C_3 : C-index of the third group (F-index is highest)

F_1 : F-index of the first group

F_2 : F-index of the second group

F_3 : F-index of the third group

$$E_0 = \frac{n+1}{n+i} (C_1 + \alpha_2 \cdot C_2 + \alpha_3 \cdot C_3) \times F_1 \qquad (5)$$

where, α_2 : (sum of the lateral shear forces sustained by the second group members corresponding to the displacement at the ultimate strength of the first group members)/(sum of the ultimate strength of the second group members) ; It may be taken as the values shown in Table 3.

α_3 : (sum of the lateral shear forces sustained by the third group members corresponding to the displacement at the ultimate strength of the first group members) / (sum of the ultimate strength of the third group members) ; It may be taken as the values shown in Table 4.

Table 3. α_2 in Eq. (5)

the first group / the second group	extremely brittle column	shear column, shear wall
flexural column	0.5	0.7
flexural wall	0.7	1.0
shear column, shear wall	0.7	——

Table 4. α_3 in Eq. (5)

the first group / the third group	extremely brittle column	shear column, shear wall
flexural column	0.5	0.7
flexural wall	0.7	1.0
shear column, shear wall	0.7	——

ii) E_0-Index of Buildings with Extremely Brittle Columns

In the case that there are some extremely brittle columns, E_0-index is the highest value wihch is calculated by Eq. (4) and (5) neglecting extremely brittle columns or by Eq. (5) considering extremely brittle columns. In the case that extremely brittle columns are not considerd, the vertical member's group, of which the ductile index is secondly least, rises to the first group, and the number of groups goes up in order.

However, if the extremely brittle columns are the secondary seismic elements, E_0-index shall be the value by Eq. (5) considering extremely brittle columns. In addition, even if the extremely brittle column is not the secondary seismic element, in the case that there are some shear columns which are the secondary seismic elements, E_0-index is the larger value which is obtained by Eq. (5) considering extremely brittle columns or by Eq. (5) neglecting extremely brittle columns.

iii) Exception

In the case that eccentricity ratio defined in Section 3.4 for the calculation of S_D-index is more than 0.15 because of unbalancedly distributed walls etc., E_0-index is the smaller one of the following two.

a) Neglecting the vertical members by which the eccentricity is caused, E_0-index is calculated by the method mentioned in Paragraph i) and ii).

b) Not considering the eccentricity, E_0-index is obtained by Eq. (5), however, the vertical members by which the eccentricity is caused is taken as the first group, and the group of which F-index is smaller than the first group is neglected.

(3) The Third Evaluation Method

The third evaluation method is performed in the same way as the second evaluation method and further the following matters are added considering the strength and ductility of beams and the rotation of foundation under wall.

i) As failure types, three other types shown in Table 5 are added to five types shown in Table 2.

ii) E_0-index is calculated in the same way as the second evaluation method, however, E_0-index may be modified as follows in only the case that the flexural yielding of beams or the overturning capacity of walls controls the seismic capacity of the building.

$$E_0' = E_0 \times \frac{2}{3} \times \frac{2n+1}{n+1} \qquad (6)$$

where, n : total number of stories of the building

Table 5. Classification of Vertical Members by Failure Types for the Third Evaluation Method

failure type	Definition
flexural column flexural wall shear column shear wall extremely brittle column	by definition in Table 2
beam yield type column	column controled by the beam that flexural yielding precedes shear failure
beam shear failure type column	column controled by the beam that shear failure precedes flexural yielding
overturning type wall	wall that overturning capacity precedes flexural yielding or shear failure

3.2.2 Strength Index, C

This section is used for calculating C-index of vertical members at each story of buildings for the first, the second and the third evaluation method.

(1) The First Evaluation Method

In the case of the first evaluation method, using the sectional area of walls and columns, strength index C is approximately calculated as follows.

$$Cw = \frac{\tau w_1}{w} \times aw_1 + \frac{\tau w_2}{w} \times aw_2 + \frac{\tau w_3}{w} \times aw_3 \qquad (7)$$

$$Cc = \frac{\tau c}{w} \times ac \qquad (8)$$

$$Csc = \frac{\tau sc}{w} \times asc \qquad (9)$$

where, Cw : strength index of walls

Cc : strength index of columns

Csc : strength index of extremely short columns

τw_1 : average shear stress at ultimate strength of wall (wall with columns on both ends) ;

30 kg/cm^2 may be used for this value.

τw_2 : average shear stress at ultimate strength of wall (wall with a column on one end) ;

20 kg/cm^2 may be used for this value.

τw_3 : average shear stress at ultimate strength of wall (wall without surrounding columns) ;

10 kg/cm^2 may be used for this value.

τc : average shear stress at ultimate strength of column ; 10 kg/cm^2 may be used for this value, however, 7 kg/cm^2 shall be used if ho/D is more than or equal to 6.

τsc : average shear stress at ultimate strength of extremely short column ; 15 kg/cm^2 may be used for this value.

aw_1 : ratio of wall sectional area to sum of floor area (wall with columns on both ends) = $Aw_1/\Sigma Af$ (cm^2/m^2)

aw_2 : ratio of wall sectional area to sum of floor area (wall with a column on one end) = $Aw_2/\Sigma Af$ (cm^2/m^2)

aw_3 : ratio of wall sectional area to sum of floor area (wall without surrounding columns) = $Aw_3/\Sigma Af$ (cm^2/m^2)

Aw_1 : sum of the effective wall sectional area in the direction at the story investigated (wall with columns on both ends) (cm^2)

Aw_2 : sum of the effective wall sectional area in the direction at the story investigated (wall with a column on one end) (cm^2)

Aw_3 : sum of the effective wall sectional area in the direction at the story investigated (wall without surrounding columns) (cm^2)

However, wall sectional area is defined by Figure 2.

ac : ratio of column sectional area to sum of floor area = $Ac/\Sigma Af$ (cm^2/m^2)

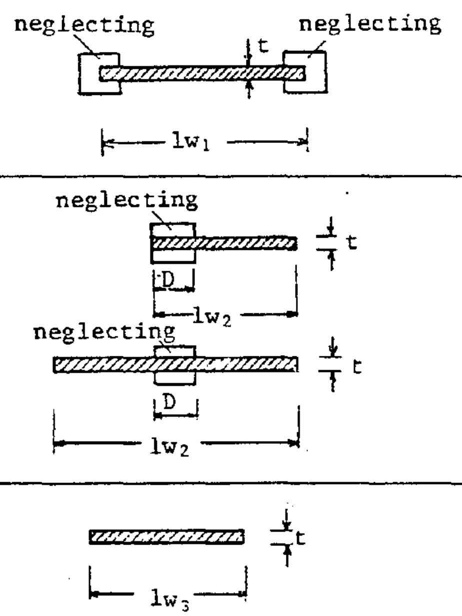
asc : ratio of extremely short column sectional area to sum of floor area = $Ac/\Sigma Af$ (cm^2/m^2)

Ac : sum of independent column sectional area at the story (cm^2) ; The column surrounding the wall which is used for the calculation of Aw_1 or Aw_2 shall not be accounted to Ac.

Asc : sum of extremely short column sectional area at the story (cm^2)

ΣAf : sum of the floor area of which the story is higher than the story calculated (m^2)

w : sum of the weight of each story which is higher than the story under consideration (dead load + live load for calculation of lateral load) $/\Sigma Af$ (kg/cm^2) ; 1,200 kg/cm^2 may be used for this value if the calculation is not especially needed.



$Aw_1 = t \times lw_1$

$Aw_2 = t \times lw_2$

If (lw_2 - D) is less than 45 cm, neglecting the wall, it may be regarded as independent column.

$Aw_3 = t \times lw_3$

If lw_3 is less than 45 cm, it is neglected.

Figure 2. Calculation of Wall Sectional Area

(2) The Second Evaluation Method

In the second evaluation method, based on the assumption that the strength of beams is in principle sufficiently large, C-index is calculated by the ultimate strength of vertical members (columns and walls) against horizontal load.

i) Process

Structural index for the second evaluation method is calculated in the following process.

a) The ultimate shear strength, Qsu and the shear force at ultimate flexural strength, Qmu of each vertical member are calculated, and then the failure types are determined by the comparison of these two values. Ultimate shear strength, Qsu and ultimate flexural strength, Mu are calculated by Eq. (10) - Eq. (15) in Paragraph ii), and shear force at ultimate flexural strength is calculated by Eq. (16) and Eq. (17) in Paragraph iii).

b) Ductility index F of each vertical member is decided by the failure type and the ductility capacity in the way of Section 3.2.3.

c) Vertical members are classified into groups (less than or equal to 3), and the structural index of each group is calculated.

Classification into groups is shown in Paragraph iv), and calculation of structural index is shown in Paragraph v).

ii) Calculation of Ultimate Strength

Ultimate flexural strength and ultimate shear strength of a member are calculated by Eq. (10) - Eq. (15).

The specified compressive strength for compressive strength of concrete (Fc), 3,000 kg/cm^2 for tensile yield stress of round bars and (specified tensile yield stress + 500 kg/cm^2) for tensile yield stress of defromed bars may be used respectively. However, in the case that remarkable time-dependent deterioration are observed by preliminaly investigation or there are data about material strength from detailed investigation, the values in the actual condition should be used.

a) Ultimate flexural strength Mu of a rectangular column is obtained by Eq. (10).

$$
\left.
\begin{array}{l}
Nmax \geqq N > 0.4\, b \cdot D \cdot Fc \\
\quad Mu = (0.8a_t \cdot \sigma_y \cdot D + 0.12b \cdot D^2 \cdot Fc) \left(\dfrac{Nmax - N}{Nmax - 0.4b \cdot D \cdot Fc} \right) \\
0.4b \cdot D \cdot Fc \geqq N > 0 \\
\quad Mu = 0.8a_t \cdot \sigma_y \cdot D + 0.5N \cdot D \left(1 - \dfrac{N}{b \cdot D \cdot Fc} \right) \\
0 > N \geqq Nmin \\
\quad Mu = 0.8a_t \cdot \sigma_y \cdot D + 0.4N \cdot D
\end{array}
\right\} \quad (10)
$$

where, Nmax : ultimate strength of the column under axial compression = $b \cdot D \cdot Fc + a_g \cdot \sigma_y$ (kg)

Nmin : ultimate strength of the column under axial tension = $-a_g \cdot \sigma_y$ (kg)

N : axial froce of the column (kg)

a_t : total area of tension bars (cm^2)

a_g : gross area of bars in the column (cm^2)

b : width of the column (cm)

D : depth of the column (cm)

σ_y : tensile yield stress of bars (kg/cm^2)

Fc : compressive strength of concrete (kg/cm^2)

b) Ultimate flexural strength Mu of a column with wing walls is calculated by Eq. (11). However, in the case that the wing wall is on only one side of the column and flexural moment acts in the direction that the wing wall is tensile, the column with the wing wall is treated as a rectangular column and is calculated by Eq. (10).

$$
N \leqq [\, 0.5\alpha_e(0.9 + \beta) - 13p_t] b \cdot D \cdot Fc
$$

$$
Mu = (0.9 + \beta)\, a_t \cdot \sigma_y \cdot D + 0.5N \cdot D \left[\, 1 + 2\beta - \frac{N}{\alpha_e \cdot b \cdot D \cdot Fc} \left(1 + \frac{a_t \cdot \sigma_y}{N} \right)^2 \right] \quad (11)
$$

If N is more than $[0.5\alpha_e(0.9 + \beta) - 13p_t]b \cdot D \cdot Fc$,

Mu is calculated by substituting $[0.5\alpha_e(0.9 + \beta) - 13p_t]b \cdot D \cdot Fc$ into N of Eq. (11).

where, p_t : tension reinforcement ratio $= a_t/(b \cdot D)$

α_e : $\Sigma A/(l_w \cdot b)$

ΣA : total sectional area of the column with wing walls (cm^2)

l_w : total horizontal length measured out-to-out of wing walls (cm)

β : (length of wing wall on compression side) / D

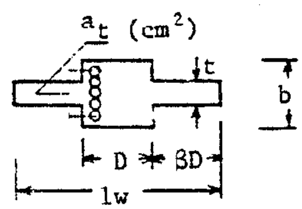


Figure 3. Column with wing walls

c) Ultimate flexural strength of a wall with columns on both ends is obtained by Eq. (12). If there are columns in the middle of the wall, the longitudinal bars of the column is regarded as vertical reinforcements of the wall.

$$Mu = a_t \cdot \sigma_y \cdot l_w + 0.5\Sigma(a_w \cdot \sigma_{wy})\ l_w + 0.5N \cdot l_w \qquad (12)$$

where, a_t : total area of longitudinal bars in the column on the tensile side of the wall (cm^2)

σ_y : tensile yield stress of longitudinal bars in the column on the tensile side of the wall (kg/cm^2)

a_w : area of vertical reinforcements in the wall (cm^2)

σ_{wy} : tensile yield stress of vertical reinforcements in the wall (kg/cm^2)

l_w : length of the wall, measured center-to-center of columns (Figure 5) (cm)

d) Ultimate flexural strength of a wall with a column on one end or a wall without columns is calculated by Eq.(10), Eq.(11) or Eq.(12) according to the shape and arrangement of reinforcing bars.

e) Ultimate shear strength of a rectangular column is calculated by Eq.(13).

$$Qsu = \left[\frac{0.053pt^{0.023}(180 + Fc)}{M/(Q \cdot d) + 0.12} + 2.7\sqrt{P_w \cdot {}_s\sigma_{wy}} + 0.1\sigma_o \right] b \cdot j \qquad (13)$$

however,

$$1 \leqq M/(Q \cdot d) \leqq 3$$

where P_t : tension reinforcement ratio (%)

P_w : shear reinforcement ratio ; In the case of $P_w \geqq 0.012$, 0.012 shall be used for P_w.

${}_s\sigma_{wy}$: tensile yield stress of shear reinforcement (kg/cm^2)

σ_o : axial stress of the column (kg/cm^2) ; In the case of $\sigma_o > 80\ kg/cm^2$, $80\ kg/cm^2$ shall be used for σ_o.

d : effective depth of the column section ; (D - 5cm) may be used for d.

M/Q : shear span ; ho/2 may be used for M/Q. ho is the clear height of the column.

j : distance between the center of tensile stress and that of compressive stress of the column section ; 0.8D may be used for j.

f) Ultimate shear strength of a column with wing walls is obtained by Eq.(14).

$$Qsu = 0.8\sqrt{Fc}\ \left(\frac{l_w}{ho}\right)\Sigma A + 0.5\left[p_w \cdot \sigma_{wy} + p_s \cdot \sigma_{sy}\ \frac{t(l_w - D)}{b \cdot D}\right] b \cdot D + 0.1N \tag{14}$$

where p_w : shear reinforcement ratio of the column

σ_{wy} : tensile yield stress of shear reinforcements (kg/cm^2)

P_s : lateral reinforcement ratio of the wing wall = $a_w/(t \cdot s)$

a_w (cm^2) is area of a set of lateral reinforcements and s (cm) is the spacing of lateral reinforcements.

σ_{sy} : tensile yield stress of lateral reinforcements (kg/cm^2)

N : axial force (kg)

ho : clear height of the column (cm)

ΣA : total sectional area (cm^2)

l_w, t, b and D is in Figure 4.

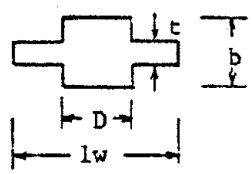


Figure 4. Column With Wing Walls

g) Ultimate shear strength of a wall with columns on both ends is calculated by Eq.(13). However, the parameters are substituted as follows. In addition, if the wall has an opening, Eq.(13) is multiplied by reduction ratio (γ) of Eq.(15).

pt : $100 \times a_t/(be \cdot l)$ (%)

where a_t : total area of longitudinal bars in the column on the tensile side of the wall (cm^2)

l : total length of the wall (Figure 5) (cm) be : equivalent thickness of wall = $\Sigma A/l$ (cm)

ΣA : total sectional area (cm^2)

P_w : equivalent holizontal reinforcement ratio of the wall = $a_w/(be \cdot s)$

where a_w : area of a set of lateral reinforcements (cm^2)

s : spacing of lateral reinforcements (cm)

${}_s\sigma_{wy}$: tensile yield stress of reinforcements of the wall (kg/cm^2)

σ_o : ΣN / $(be \cdot l)$

where ΣN : total axial force (kg)

j : l_w or 0.8l may be used for this value.

b : It is replaced by be.

D : It is replaced by l.

d : It is replaced by l.

M/Q : wMu / wQmu calculated by Eq.(17)

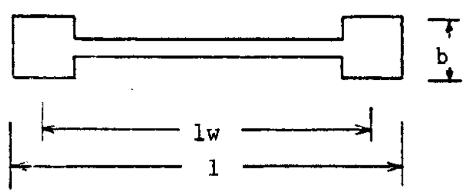


Figure 5. Wall with Columns on Both Side

reduction ratio by a opening of the wall :

$$\gamma = 1 - \text{(equivalent opening peripheral ratio)} \qquad (15)$$

where equivalent opening peripheral ratio :

$$\sqrt{\frac{\text{opening area}}{h \times l_w}}$$

h : height of the story

h) Ultimate shear strength of a wall with a column on one side or a wall without columns is calculated by Eq.(13) or Eq.(14) according to the shape and arrangement of reinforcing bars.

iii) Calculation of Failure Type and Shear Force at Ultimate Strength

Uding ultimate flexural strength and ultimate shear strength in Paragraph ii), the failure type of vertical members and shear force at the ultimate strength are obtained as follows.

a) Column

Calculating shear force ${}_cQ_{Mu}$ at ultimate flexural strength by Eq.(16), and comparing ${}_cQ_{Mu}$ with ultimate shear strength ${}_cQ_{su}$, the failure type and the shear force ${}_cQ_u$ at ultimate strength are obtained.

1) In the case of ${}_cQ_{Mu} < {}_cQ_{su}$, failure type is flexural column. $({}_cQ_u = {}_cQ_{Mu})$

2) In the case of ${}_cQ_{Mu} \geqq {}_cQ_{su}$, failure type is shear column. $({}_cQ_u = {}_cQ_{su})$

However, in shear columns, the column, that h_o/D is less than or equal to 2, is especially treated as extremely brittle column.

$$ {}_cQ_{Mu} = \frac{({}_cM_u)T + ({}_cM_u)B}{h_o} \tag{16} $$

where $({}_cM_u)T$: ultimate flexural strength at the top of the column

$({}_cM_u)B$: ultimate flexural strength at the bottom of the column

h_o : clear height of the column

b) Wall

Calculating shear force ${}_wQ_{Mu}$ at ultimate flexural strength by Eq.(17), and comparing ${}_wQ_{Mu}$ with ultimate shear strength ${}_wQ_{su}$, the failure type and the shear force ${}_wQ_u$ at ultimate strength are obtained.

3) In the case of ${}_wQ_{Mu} < {}_wQ_{su}$, failure type is flexural wall.

$({}_wQ_u = {}_wQ_{Mu})$

4) In the case of ${}_wQ_{Mu} \geqq {}_wQ_{su}$, failure type is shear wall.

$({}_wQ_u = {}_wQ_{su})$

$$ {}_wQ_{Mu} = 2 \cdot {}_wM_u / h_w \qquad (17) $$

However, in the case of the top story of a multistoried wall (including a single storied wall), the coeffeicient 2 of right side in Eq.(17) is replaced by 1.

where ${}_wM_u$: ultimate flexural strength of the wall at the story under consideration

h_w : total height of the wall measured from the floor considerd to the top

c) By the above calculation, the failure type of each vertical member is any one in Table 6.

Table 6. Failure Types and Ductility Index

(The Second Evaluation Method)

failure type	F-index (Section 3.2.3)
1) flexural column	calculated from ductility factor μ at ultimate strength (1.27 -3.2*)
2) flexural wall	calculated from the ratio of shear strength to flexural strength (1.0 -2.0)
3) shear column	1.0
4) shear wall	1.0
5) extremely brittle column	0.8

* There is the case that F-index is equal to 1.0 according to the particular condition as shown in Eq.(23).

iv) Classification of Vertical Members

Based on the failure types decided in the above Paragraph iii) and the values of F-index calculated in Section 3.2.3, vertical members are classified into three or less groups, each group is named for the first, the second and the third group.

In this case, the following matters are important.

a) Collecting the members of which the F-index are near each other into one group, the number of groups shall be reduced as possible. In this case, the minimum value of the F-indexes of the members in the group is used for the F-index of the group.

b) Extremely brittle columns should be an independent group.

v) Calculation of Strength Index

Strength index Ci of each group is obtained by Eq.(18).

Ci = (sum of the shear force at ultimate strength of the vertical members belonging to the i-th group) /ΣW (18)

where ΣW : sum of the weight of which the story is higher than the story under consideration (dead load + live load for calculation of lateral load)

(3) The Third Evaluation Method

i) Process

Structural index for the third evaluation method is calculated in the following process.

a) Ultimate flexural strength Mu and ultimate shear strength Qsu of columns, walls and beams are calculated according to the way shown in Paragraph ii).

b) Using the result of a), failure type of each member and ultimate moment of each nodel point is determined, and then, failure type and lateral shear force of vertical member are calculated by a nodal limit analysis. However, walls are calculated by an approximate limit analysis assuming the distribution of lateral load and the failure mechanism.

c) In the same way as the second evaluation method, vertical members at each story are classified into three or less groups, and strength index of each group is calculated.

ii) Calculation of Ultimate Strength of Members

a) Ultimate flexural strength and ultimate shear strength of walls and columns are obtained by Eq.(10) -Eq.(15) in the same manner as the second evaluation method.

b) Ultimate flexural strength and ultimate shear strength of beams are calculated by Eq.(10) -Eq.(15) respectively substituting N=0 or $\sigma_0=0$ into these equations. However, for the calculation of ultimate flexural strength of beams, the following Eq.(19) is also applicable. In addition, the effect of the reinforcements arranged in slabs and the effect of the bars at the middle depth of beams may be considered.

$$Mu = 0.9a_t \cdot \sigma_y \cdot d \quad (19)$$

where, a_t : total area of tensile bars (cm^2)

σ_y : tensile yield stress of tensile bars (kg/cm^2)

d ; effective depth of a beam cross-section (cm)

iii) Determination of Failure Type and Lateral Shear Force at Ultimate Strength

a) columns

Considering the case that the lateral capacity of columns depends on the ultimate strength of beams, failure types and lateral shear force of columns at ultimate strength are determined by nodal limit analysis.

1) Failure types of beams and columns are determined in the same way as the second evaluation method, and the end moments of members at the nodal points are calculated.

2) According to Figure 6, comparing the sum of the end moments of beams with that of columns at each nodal point, if the sum of the end moments of beams is less than that of columns, each half of it is used for the ultimated end moment of the upper and lower column at the nodal point. If the sum of the end moments of columns is less than that of beams, they are used for the ultimate end moments of columns as they are. In this case, the failure type and F-index of the member that controls the ultimate condition of the nodal point are used for the failure type and F-index of the nodal point.

3) After calculating failure types and ultimate end moments about all nodal points, the failure type and lateral shear force at ultimate strength of a column are determined as follows.

Failure type of column : the failure type of the nodal point of which F-index is lower comparing the two nodal points at top and bottom end of the column.

Lateral shear force ${}_cQ_u$ at ultimate strength of column :
${}_cQ_u$ = (sum of ultimate moments at top and bottom end of the column)/(clear height of the column)

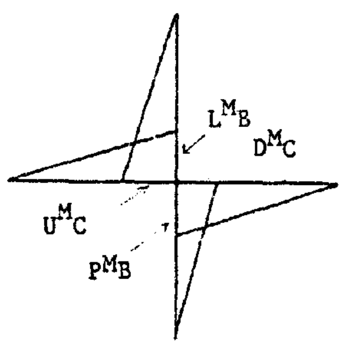


Figure 6. Failure Type of Nodal Point

note 1) In the case of $({}_uM_c + {}_bM_c) > ({}_LM_B + {}_RM_B)$, $\frac{1}{2}({}_LM_B + {}_RM_B)$ is used for each ultimate end moment of columns at the nodal point. The failure type of beams is used for the failure type of the nodal point.

note 2) In the case of $({}_{U}M_C + {}_{D}M_C) \leqq ({}_{L}M_B + {}_{R}M_B)$, ${}_{U}M_C$ and ${}_{D}M_C$ are used for ultimate end moments of columns at the nodal point as they are.

note 3) ${}_{U}M_C$, ${}_{D}M_C$, ${}_{L}M_B$ and ${}_{R}M_B$ are calculated considering the effect of the rigid zone.

b) Walls

As shown in Figure 7, multi-storied wall is idealized by cutting off from the other framing members at the mid-span of connecting beams. The lateral load applied to the idealized wall may be taken as the least value of the following three lateral loads determined uner inverse triangular distribution of lateral loads ; the lateral load by which the wall reaches to their flexural yield strength, shear failure strength or overturning capacity. Lateral shear force at ultimate strength of the wall at each story is calculated from the above lateral load, and this failure type is used for the failure type of the wall at each story.

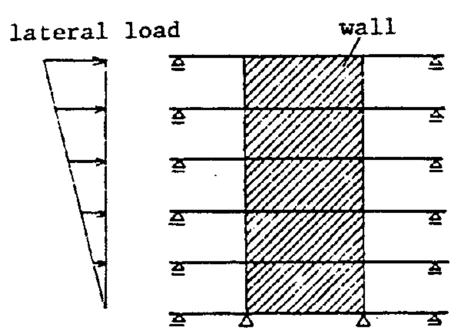


Figure 7. Multi-Storied Wall

The wall which is not multi-storied is treated in the same way as multi-storied wall, based on an assumption of the failure mechanism that is as actual as possible.

By the above calculation, the failure type of each vertical member is any one in Table 7.

Table 7. Failure Types and Ductility Index

(The Third Evaluation Method)

failure type	F-index (Section 3.2.3)
1) flexural column	F-index in Table 6
2) flexural wall	F-index in Table 6
3) shear column	F-index in Table 6
4) shear wall	F-index in Table 6
5) extremely brittle column	F-index in Table 6
6) beam yield type column	3.0
7) beam shear failure type column	1.5
8) overturing type wall	3.0

iv) Classification of vertical members based on failure types and ductility indexes, and calculation of strength indexes of the groups are performed in the same manner as the second evaluation method.

3.2.3 Ductility Index, F

1) Calculation of F-index

F-index of vertical members is calculated as follows according to the number of the evaluation method and failure type of the member determined in Section 3.2.2.

i) The First Evaluation Method

Following the classification of vertical members shown in Table 1, F-index shown in Table 8 is used in the first evaluation method.

Table 8. Ductility Index

(The First Evaluation Method)

name	F-index
column ($h_o/D > 2$)	1.0
extremely short column ($h_o/D \leqq 2$)	0.8
wall	1.0

ii) The second Evaluation Method

Following the classification of vertical members shown in Table 2, F-index shown in Table 9 is used in the second evaluation method. Here, F-indexes of flexural columns and flexural walls are obtained respectively by Eq.(20) and Eq.(21) because of their well ductility. However, F-index of columns with wing walls is taken as equal to 1.0.

a) flexural columns

$$F = \phi\sqrt{2\mu - 1} \qquad (20)$$

where, μ : ultimate ductility factor, calculated by Eq.(22)

ϕ : $\dfrac{1}{0.75(1 + 0.05\mu)}$

b) flexural walls

$$\begin{aligned} & {}_wQ_{su} / {}_wQ_u \leqq 1.3 && ;\ F = 1.0 \\ 1.3 < & {}_wQ_{su} / {}_wQ_u < 1.4 && ;\ F = -12.0 + 10 \times ({}_wQ_{su} / {}_wQ_u) \qquad (21) \\ 1.4 \leqq & {}_wQ_{su} / {}_wQ_u && ;\ F = 2.0 \end{aligned}$$

where, ${}_wQ_{su}$: ultimate shear strength of the wall

${}_wQ_u$: shear force at ultimate strength (at ultimate flexural strength) of the wall

iii) The Third Evaluation Method

In the same way as the second evaluation method, F-index is determined according to Table 9. However, following the classification shown in Table 5, the latter articles of Table 9 are also applied.

Table 9. Ductility Index (The Second and The Third Evaluation Method)

failure type	F-index	evaluation method
flexural column	calculated by Eq.(20) 1.27 - 3.2*	second, third
flexural wall	calculated by Eq.(21) 1.0 - 2.0	" , "
shear column	1.0	" , "
shear wall	1.0	" , "
extremely brittle column	0.8	" , "
beam yield type column	3.0	third
beam shear failure type column	1.5	"
overturning type wall	3.0	"

* There is the case that F-index is equal to 1.0 according to the particular condition as shown in Eq.(23).

(2) Determination of Ultimate Ductility Factor μ of Flexural Columns

Ultimate ductility factor μ of flexural columns is obtained by Eq.(22). However, if any one of the conditions described in Eq.(23) is corresponded, the value of F-index should be 1.0.

$$\mu = \mu o - k_1 - k_2 \qquad (1 \leqq \mu \leqq 5) \qquad (22)$$

where $\mu o = 10 \cdot ({}_cQ_{su} \,/\, {}_cQ_u - 1)$

$k_1 = 2.0$ (k_1 may be zero provided that shear reinforcement spacing is less than eight times the diameter of longitudinal bars.)

$k_2 = 30\left(\frac{{}_c\tau_u}{F_c} - 0.1 \right) \geqq 0$

$_cQ_{su}$: ultimate shear strength of the column

$_cQ_u$: lateral shear force of the column at the ultimate condition

$c\tau u$: $_cQ_u/(b \cdot j)$

b : width of the column

j : distance beween the center of tensile stress and that of compressive stress of the column section; 0.8D may be used for it.

F_c : compressive strength of concrete

Conditions in which F-index should be taken as 1.0 ;

$$\left.\begin{array}{l} N_s/(b \cdot D \cdot F_c) > 0.4 \\ c\tau u/\ F_c > 0.2 \\ p_t > 1\% \\ h_o/\ D \leqq 2 \end{array}\right\} \quad (23)$$

where, N_s : axial force of the column at the failure mechanism

p_t : tensile reinforcement ratio of the column section

h_o : clear height of the column

3.3 Seismic Sub-Index of Ground Motion, G

G-index may be taken as equal to 1.0 at present.

3.4 Seismic Sub-Index of Structural Profile, S_D

3.4.1 General

This index quantitatively represents the effect of the structural profile, the distribution of stiffness etc. on the seismic safety of buildings, and is used to modify E_o-index.

S_D-index is determined for two method, the first and the second evaluation method, according to the required accuracy.

3.4.2 Judgement Items

Items applied in each method are as follows.

(1) Items in The First Evaluation Method

i) Items Concerning Floor Plan Profile

Irregurality of Plan, length-width ratio in plan, dent in plan, clearance of expansion joints, presence of open hall (the size and eccentricity) and other special profiles in plan.

ii) Items Concerning Sectional Profile

Presence of underground stories, uniformity of story height, presence of piloti and other special profiles in section.

(2)Items in The Second Evaluation Method

In the second evaluation method, the following items are examined in addition to the items considered in the first evaluation method.

i) Items Concerning Horizontal Rigidity

Eccentricity between the center of gravity and the center of rigidity in plan.

ii) Items Concerning Sectional Rigidity

Weight-stiffness ratio of a story to that of the immidiately above story.

3.4.3 Calculation of S_D-index

The influence factor qi, which represents the degree of influence of each judgement item, is calculated using the grading factor Gi and the adjusting factor Ri for the range of the influence. Then S_D-index is obtained by the mutual multiplication of qi as shown in Eq.(24) and Eq.(25).

The degree of influence is adjusted according to the classification shown in Table 10, using respectively R1i and R2i in the first and the second evaluation method.

(1) Equation To Be Used in The Caluculation of S_D-Index

i) S_D-index for The First Evaluation Method

$$S_D = q_{1a} \times q_{1b} \times \cdots\cdots \times q_{1k} \qquad (24)$$

where, $q_{1i} = [\ 1 - (1 - Gi) \times R_{1i}\ ]$

$(i = a,b,c,d,e,f,g,i,j,k)$

$q_{1i} = [\ 1.2 - (1 - Gi) \times R_{1i}\ ]$

$(i = h)$

ii) S_D-Index for The Second Evaluation Method

$$S_D = q_{2a} \times q_{2b} \times \cdots\cdots \times q_{2k} \qquad (25)$$

where, $q_{2i} = [\ 1 - (\ - Gi) \times R_{2i}\ ]$

$(i = a,b,c,d,e,f,g,i,j,k,l,m,n,o)$

$q_{2i} = [\ 1.2 - (1 - Gi) \times R_{2i}\ ]$

$(i = h)$

iii) S_D-Index for The Third Evaluation Method

S_D-indexes for the second evaluation method are used for the third evaluation as they are.

$$S_{D3} = S_{D2}$$

(2) Classification of Items

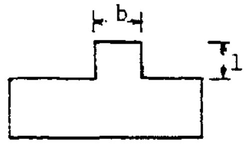
The classification of items and the values of G-factors and R-factors are shown in Table 10.

Table 10. Classification of Items and List of G and R-factor

Items				Gi 1.0	Gi 0.9	Gi 0.8	R: R1i	R: R2i	application
the 1st and the 2nd evaluation method (1,2)	floor plan profile (P)	a	irregularity	regular a_1	middle a_2	irregular a_3	1.0	0.5	
		b	length-width ratio	$b \leqq 5$	$5 < b \leqq 8$	$8 < b$	0.5	0.25	
		c	dent	$0.8 \leqq c$	$0.5 \leqq c < 0.8$	$c < 0.5$	0.5	0.25	These items are applied to all of the stories using the factors which are gained about the most disadvantageous story.
		d	clearance of expansion joints*1	$\frac{1}{100} \leqq d$	$\frac{1}{200} \leqq d < \frac{1}{100}$	$d < \frac{1}{200}$	0.5	0.25	
		e	size of open hall	$e \leqq 0.1$	$0.1 < e \leqq 0.3$	$0.3 < e$	0.5	0.25	
		f	eccentricity of open hall	$f_1 \leqq 0.4$ and $f_2 \leqq 0.1$	$f_1 \leqq 0.4$ and $0.1 < f_2 \leqq 0.3$	$0.4 < f_1$ or $0.3 < f_2$	0.25	0	
		g	other special profile*2				0.5	0.25	
	sectional profile (S)	h	underground stories	$1.0 \leqq h$	$0.5 \leqq h < 1.0$	$h < 0.5$	1.0	1.0	
		i	uniformity of story height	$0.8 \leqq i$	$0.7 \leqq i < 0.8$	$i < 0.7$	0.5	0.25	
		j	presence of piloti	no	all of the floor	uneven distribution of piloti	0.5	0.25	
		k	other special profile*3				0.5	0.25	
the 2nd evaluation method (2)	horizontal rigidity (RP)	l	eccentricity between the center of gravity and the center of rigidity	$l \leqq 0.1$	$0.1 < l \leqq 0.15$	$0.15 < l$		1.0	These items are applied to each story about the longitudinal and rigid directions.
		m							
	sectional rigidity (SR)	n	weight-stiffness ratio of a story to that of the immidiately above story	$0.8 \leqq n \leqq 1.2$	$1.2 < n \leqq 1.7$ or $0.6 \leqq n < 0.8$	$1.7 < n$ or $n < 0.6$		1.0	
		o							

Notes concerning Table 10;

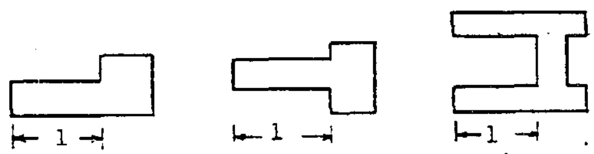
a_1 : The plan is almost symmetric about each direction, and the area of a lump is less than or equal to ten percent of the floor area. Lumps are considered in the case of $l/b \geqq 1/2$.



a_2 : The plan is more irregular than that of a_1, and the area of a lump is less than or equal to thirty percent of the floor area in the plan of L-type, T-type, U-type and others.

a_3 : The plan is more irregular than that of a_2, and the area of a lump is more than thirty percent of the floor area in the plan of L-type, T-type, U-type and others.

b : b = (length of the long side)/(length of the short side) ;
In the plan of L-type, T-type, U-type and others, $2 \cdot l$ is used for the length of the long side.



c : $c = D_1/D_0$

d : This is applied to the buildings which have expansion joints.
d = (clearance of expansion joints)/(height of the part connected by expansion joints)

e : e = (area of open hall)/(area of the floor including the area of open hall) ; However, a stair hall surrounded in reinforced concrete walls is not regarded as an open hall.

f : f_1 = (distance between the center of the plan and the center of the open hall)/(length of the short side)

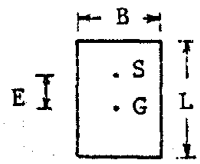
f_2 = (distance between the center of the plan and the center of the open hall)/(length of the long side)

h : h = (area of basement floor)/(building area)

i : i = (height of the immediately above story)/(height of the story under consideration) ; When the top story is examined, the immediately above story in this equation is replaced with the immediately below story.

j : In the case that the floor is supported by only piloti, moreover, the distribution of piloti is eccentric, it is treated as eccentric distribution. When the building is complete flaming structure, however, it is not considered as piloti.

l : $l = E/\sqrt{B^2 + L^2}$



S : center of gravity

G : center of rigidity

Here, horizontal rigidity of each plan may be obtained by [Σ(column sectional area) + α X Σ (wall sectional area)]*4 of each plane.

n : n = [(weight-stiffness ratio of the immediately above story)/(weight-stiffness ratio of the story under consideration) X β ; When the top story is examined, the immediately above story in this equation is replaced with the immediately below story.
Where,(weight-stiffness ratio) = (rigidity at the story under consideration)/(sum of the weight at the higher stories than the story under consideration), (rigidity at the story) = [Σ(column sectional area) + Σ(wall sectional area) X α]/(height of the story), $\beta = (N - 1)/N$.

N : the number of the stories above the story under consideration, $\beta = 2.0$ at the top story.

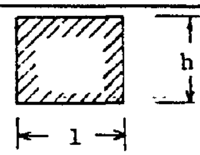
*1 : In the case that expansion joints are utilized in the building, each part divided by expansion joints is considered as one unit.

*2 : This item is used in the case that the plan is remarkably special profile.

*3 : This item is used in the case that the section is remarkably special profile.

*4 : The value of α is variable according to (height of the wall)/(length of the wall).

h/l	α	
	wall surrounded by framing members	wall not surrounded by framing members
$3.0 \leqq h/l$	1.0	0.3
$2.0 \leqq h/l < 3.0$	1.5	0.5
$1.0 \leqq h/l < 2.0$	2.5	0.8
$h/l < 1.0$	3.5	1.2



3.5 Seismic Sub-Index of Time-Depended Deterioration, T

3.5.1 General

T-index aims to evaluate the effect of the structural defects, such as cracks, deflections, superannuations and others on the seismic safety of buildings. Therefore, determination of T-index should be performed essencially according to the detailed site investigation. However, considering the conveniency of this evaluation method and the accuracy about the other sub-indexes (E_O-index, S_D-index and others) used in the calculation of seismic index of structure, I_s, investigation method is classified into three steps, namely the first investigation, the second investigation and the third investigation. T-index is determined in principle according to these investigations and is respectively used at the calculation of I_s-index in the first, the second and the third evaluation method.

3.5.2 The First Evaluation Method

T-index for the first evaluation method is determined following the result of the first investigation shown in Table 11. The minimum value of the T-values in C column of Table 11 is used for T-index of the first evaluation method.

3.5.3 The Second Evaluation Method

T-index for the second evaluation method is calculated by Eq.(26) in accordance with the resultant of the second investigation shown in Table 12.

$$\left.\begin{aligned} T &= (T_1 + T_2 + T_3 + \cdots\cdots\cdots\cdots + T_N)/N \\ T_i &= (1 - P_{si})(1 - P_{ti}) \end{aligned}\right\} \qquad (26)$$

where T_i : T-index of i-story

N : number of stories examined

P_{si} : sum of the demerit points at i-story concerning about structural cracks and deflections. However, it may be taken as equal to zero if the investigation is not needed.

P_{ti} : sum of the demerit points at i-story concerning about deterioration and superannuation. However, it may be taken as equal to zero if the investigation is not necessary.

3.5.4 The Third Evaluation Method

In the third evaluation method, the same value of T-index as the value determined in the second evaluation may be used in principle. However, in the case that C-index is calculated using the result of detailed investigation, T-index may be taken as 1.0.

3.5.5 Investigation of Buildings

(1) The First Investigation

The first investigation is performed about the checking items shown in Table 11 according to the explanation by the building manager and the site observation by the investigator.

(2) The Second Investigation

The second investigation is in principle examined on the following matters according to the observation of the building surface and brief measurement by the investigators. However, in accordance with the degree of the cracks and deterioration, the following matters are investigated after taking away a part of finished materials.

i) the degree and extent of structural cracks and deflections

ii) the degree and extent of deterioration and superannuation. This investigation is in principle performed about the degree and extent of several items shown in Table 12 at each story. However, the story impossible to be examined is neglected.

3) Detailed Investigation

In the case that drawing and specification have the defects, detailed investigation is performed about the following items concerning columns, beams and walls. In order to gain the information for the modification and supplement of the data that is necessary to calculate E_o-index, test pieces are extracted from the structure, a part of finish materials is taken away, a part of concrete is chipped and so on.

i) strength and elastic modulus of concrete

ii) confirmation about arrangements and sections of reinforcements

iii) reestimation of the sectional capacity of members considering the construction condition, cracks and loss

iv) reestimation of the strength of materials considering the neutralization and superannuation of concrete and the rust of reinforcements

Table 11. Calculation Table of T-Index in The First Investigation

A checking items	B degree	C T-value mark the corresponding matter	D items related to the second investigation
deflection	The building is inclined, or unequal settlement has arisen undoubtedly.	0.7	structural cracks and deflection
	The site is reclaimed ground or rice field before.	0.9	
	The deflection of beams and columns is visible with the unaided eye.	0.9	
	The above matters do not correspond to the building	1.0	
cracks of walls and columns	The leakage of rain water is observed and reinforcement is in rust.	0.8	structural cracks and deflection
	The inclined cracks of columns are clearly visible with the unaided eye.	0.9	
	There are numbers of cracks in the external wall.	0.9	
	The leakage of rain water is observed, but reinforcement is not in rust.	0.9	
	The above matters do not correspond to the building	1.0	

fire damage	There is the trace of fire damage.	0.7	structural cracks, deflection, deterioration and superannuation
	The building was damaged by fire, but the trace is not clear.	0.8	
	no experience	1.0	
usage	Chemicals were or have been used.	0.8	deterioration and superannuation
	The above matter does not correspond.	1.0	
years elapsed	more than or equal to thirty years	0.8	deterioration and superannuation
	more than or equal to twenty years	0.9	
	less than twenty years	1.0	
condition of finish materials	The separation of external finish materials is remarkable by the superannuation	0.9	deterioration and superannuation
	The deterioration or separation of internal finish materials is remarkable	0.9	
	There is no particular trouble	1.0	

Table 12. Sum Up Table of Demerit Points in The Second Investigation (floor)

note : Mark the corresponding matters and then sum up them.

members \ degree \ Extent (items)		structural cracks and deflection a: 1. cracks following uneven settlement 2. shear cracks or inclined cracks of beams, walls and columns clearly visible with the unaided eye	b: 1. deflection of slabs and beams interfering with non-structural elements 2. shear cracks or inclined cracks of beams, walls and columns not clearly visible with the unaided eye 3. flexural cracks or vertical cracks of beams and columns clearly visible with the unaided eye	c: 1. brief structural cracks not corresponding with a or b 2. deflection of slabs and beams not corresponding with a or b
I floor	i	0.017	0.005	0.001
	ii	0.006	0.002	0
	iii	0.002	0.001	0
II beam (girder)	i	0.050	0.015	0.004
	ii	0.017	0.005	0.001
	iii	0.006	0.002	0
III wall, cloumn	i	0.150	0.046	0.011
	ii	0.050	0.015	0.004
	iii	0.017	0.005	0.001
sum of demerit points	sub-total			
	total	$p_s =$		

i, ii, iii denotes more than 1/3 of total number of floors, from 1/3 to 1/9 and less than 1/9, respectively.

deterioration and superannuation		
a	b	c
1. expansion cracks of concrete by the rust reinforcement 2. corrosion of reinforcement 3. cracks by fire 4. deterioration of concrete by chemicals and so on	1. melt of rust on reinforcement by leakage water 2. neutralisation of concrete to the place of reinforcement 3. remarkable separation of finish materials	1. remarkable dirt or stain by leakage water and chemicals and so on 2. brief separation or superannuation of finish materials
0.017 0.006 0.002	0.005 0.002 0.001	0.001 0 0
0.050 0.017 0.006	0.015 0.005 0.002	0.004 0.001 0
0.150 0.050 0.017	0.046 0.015 0.005	0.011 0.004 0.001
p_t =		

4. Calculation of Seismic Index of Non-structural Index, I_N

4.1 General Rule

Seismic Index of non-structural elements, I_N is an index evaluating the safety against the injury of non-structural members, especially considering the injury thar the separation and fall of finish materials on external walls by earthquake injure people directly or disturb their refuge.

The evaluation is composed of the first, the second and the third evaluation method, and I_N-index is calculated about each wall surface at each story in all methods.

4.2 The First Evaluation Method

4.2.1 General

In the first evaluation method, I_N-index is obtained by Eq.(27) about each wall surface at each story of buildings.

$$I_N = 1 - B \cdot H \qquad (27)$$

where, B : sub-index of structural type

H : sub-index of degree of influence

For B and H-index in Eq.(27), the values of the rectangular part including the structural type that will be destroyed earliest (B-index is the highest) at the wall surface under consideration are adopted.

4.2.2 Sub-Index of Structural Type, B

B-index is obtained by Eq.(28) using sub-index of flexibility, f and sub-index of actual consition, t.

$$B = f + (1 - f) \cdot t \qquad (28)$$

(1) Sub-Index of Flexibility, f

f-index is gained by Table 13 using grade of flexibility of structures, g_s and grade of flexibility of non-structural elements, g_N.

g_s and g_N are shown respectively in Table 14 and 15.

Table 13. Sub-Index of Flexibility, f

non-structural elements \ structure		rigid ← g_s → flexible I	II
rigid ↑ g_N ↓ flexible	I	0.5	1.0
	II	0	0.5

Table 14. Grade of Flexibility of Structures, g_s

g_s	condition of structure
rigid I	Ductility capacity is low. For instance, the building with many short columns.
↕	
flexible II	Ductility capacity is high. For instance, the building with little walls.

Table 15. Grade of Flexibility of Non-Structural Elements, g_N

g_N	non-structural elements
rigid I	Deflection capacity is low. For instance, concrete block, glass block, fixed sash window, stone facing, tile facing, mortar plastering, ALC boad and so on.
↕	
flexible II	Deflection capacity is high. For instance, metal and PC curtain wall, movable sash, stray and placing tile, naked concrete and so on.

(2) Sub-Index of Actual Condition, t

t-index is obtained by Table 16 in accordance with the existance of trouble experience.

Table 16. Sub-Index of Actual Condition, t

trouble experience	t
exist or unknown	1.0
no	0.5

4.2.3 Sub-Index of Degree of Influence, H

H-index is obtained by Table 17 according to the environment directly below the wall surface and the existance of suppression matters such as eaves, set back and others.

Table 17. Sub-Index of Degree of Influence, H

environment	suppression matters	
	exist	no
road (including private road, public square and others)	1.0	0.3
the others	0.5	0.1

4.3 The second Evaluation Method

4.3.1 General

In the second evaluation method, I_N-index is calculated by Eq.(29) about each wall surface at each story of buildings.

$$I_N = 1 - \frac{\sum_j B_j \cdot W_j \cdot H_j \cdot L_j}{\sum_j L_j} \qquad (29)$$

where, B_j : sub-index of structural type

W_j : sub-index of wall surface area

H_j : sub-index of degree of influence

L_j : length of unit of wall surface

In the application of Eq.(29), the wall surface is devided into units (rectangular parts) in the horizontal direction. The total sign Σ in Eq.(29) represents the total of these units.

In addition, in the case that a unit consists plural structural types, the structural type considered to be destroyed earliest (B-index of it is the highest) stands for the unit.

4.3.2 Sub-Index of Structural Type, B

B-index is obtained by Eq.(30) using sub-index of flexibility, f and sub-index of actual condition, t.

$$B = f + (1 - f)t \tag{30}$$

(1) Sub-Index of Flexibility, f

f-index is obtained by Table 18 using grade of flexibility of structures, g_s and grade of flexibility of non-structural elements, g_N.

g_s and g_N are shown respectively in Table 19 and 20.

Table 18. Sub-Index of Flexibility, f

structure / non-structural elements		rigid ← g_s → flexible 1	2	3	4
rigid	1	0.3	0.8	0.9	1.0
↑	2	0	0.3	0.8	0.9
g_s ↓	3	0	0	0.3	0.8
flexible	4	0	0	0	0.3

Table 19. Grade of Flexibility of Structures, g_s

g_s		condition of structure	approximate F-index
rigid	1	Ductility capacity is low. For instance, the building that extremely brittle columns nearly determine the seismic capacity.	0.8
	2	Ductility capacity is rather low. For instance the building that shear columns or shear walls nearly determine the seismic capacity.	1.0
	3	Ductility capacity is rather high. For instance, the building that flexural columns or flexural walls nearly determine the seismic capacity.	1.3
flexible	4	Ductility capacity is high. For instance, the building that flexural walls nearly determine the seismic capacity and that is especially ductile.	3.0

Table 20. Grade of Flexibility of Non-Structural Elements, g_N

<table>
<tr><th colspan="2">g_N</th><th colspan="3">non-structural elements (examples of walls, openings and external finish materials)</th></tr>
<tr><td rowspan="8">rigid
↑
↓
flexible</td><td rowspan="2">1</td><td colspan="3">Deflection capacity is low ; wet system</td></tr>
<tr><td>concrete block, glass block</td><td>fixed sash window (steel sash)</td><td>stone facing</td></tr>
<tr><td rowspan="2">2</td><td colspan="3">Deflection capacity is rather low ; dry system</td></tr>
<tr><td>ALC boad</td><td>fixed sash window</td><td>tile facing, mortar plastering</td></tr>
<tr><td rowspan="2">3</td><td colspan="3">Deflection capacity is rather high ; elements monolithic with walls placing in site ; prefabricated elements</td></tr>
<tr><td>metal or PC curtain wall</td><td>movable sash</td><td>spray or placing tile</td></tr>
<tr><td rowspan="2">4</td><td colspan="3">There are no elements which easily separate or fall ; sufficient consideration against earthquake</td></tr>
<tr><td>monolithic wall in site</td><td>(no openings)</td><td>no finish materials</td></tr>
</table>

(2) Sub-Index of Actual Condition, t

t-index is obtained by Table 21 in accordance with the combination of g_H and g_Y. g_H and g_Y are respectively grade of the trouble history of non-structural elements and grade of years passed.

Table 21. Sub-Index of Actual Condition, t

trouble history and the grade, g_H \ Passed years and the grade, g_Y		1 less than 3 years	2 3 - 10 years	3 more than 10 years
1	The building has an experience of trouble, but it is not repaired.	1.0	1.0	1.0
2	The trouble history of the building is unknown.	0.2	0.3	0.5
3	The building has no experience of trouble, or it was repaired entirely.	0	0.2	0.3

4.3.3 Sub-Index of Wall Surface Area, W

W-index is calculated by Eq.(31).

$$W = a + b\,\frac{h_j}{h_s} \tag{31}$$

where, $a = 0.5$

$b = 0.5$

h_j : height of corresponding structural type

h_s : standardized height = 3.5 m

4.3.4 Sub-Index of Degree of Influence, H

H-index is gained by Eq.(32) using sub-index of environments, e and sub-index of the arrest of falls, c.

$$H = \sum_k e_K \cdot c_K \tag{32}$$

In the application of Eq.(32), e_K and c_K are gained from every horizontal surface which is inside of the influence angle (the angle between the wall surface and the inclined plane with inclination of 1/2 from the top of the wall), and they are sumed up. However, when the kinds of e_K or c_K are more than two in a horizontal surface, the maximun value of them is used in the surface.

(1) Sub-Index of Environments, e

e-index is obtained by Table 22 in accordance with the environment (the possibility of people's being there) directly blow the wall surface.

Table 22. Sub-Index of Environment, e

environments	e
public road	1.0
private road, road in site, corridor, public square, veranda	0.7
open space where people may come, plantation	0.2
open space where people may not come, adjacent building	0

(2) Sub-Index of The Arrest of Falls, c

c-index is obtained by Table 23 according to the existance of suppression matters such as eaves, set back and so on or the other conditions.

Table 23. Sub-Index of The Arrest of Falls, c

suppression matters	c
the case that the influence angle is entirely intercepted by eaves, set back and so on	0
the pojected horizontal surface directly below the eaves that partially intercept the influence angle	0
the horizontal surface at the same story as that of the walls considered	0.5
the others	1.0

4.4 The Third Evaluation Method

In the calculation of sub-index of structural type, the practical investigation about the actual condition of the structural type (detail, state of construction, degree of superannuation and so on; they influence the deflection capacity) is performed, and then based on the resultant, the way in the second evaluation method is applied.

5. Synthetic Evaluation of Seismic Safety

Using the above mentioned I_S-index and I_N-index, the seismic safety of buildings should be evaluated synthetically.

Based on the result of the evaluation, in addition, taking account of various conditions such as the use, importance and age of buildings, judgement of seismic safety of buildings are performed according to the judgement standard that is established elsewhere. Therefore, it is desirable to make the evaluation list (the karte) clearly sta-ed the number of the evaluation, the items of I_s and I_N-index, the opinion about the result of evaluation and others.

SEISMIC RESISTANCE OF INTERIOR PARTITIONS

PRESENTED BY
M.S. AGBABIAN

1. INTRODUCTION

The primary targets of seismic hazard reduction measures in Los Angeles are unreinforced masonry buildings that normally have a high occupancy load, such as apartment houses, hotels, nursing homes, and office buildings. A common characteristic of these facilities is the extensive use of floor-to-ceiling interior walls required to partition off floor space. The capacity of these partitions to function as shear walls is recognized in Division 68 of the new Los Angeles Municipal Code (Ref. 1) in that "existing materials including wood shear walls may be used as part of the lateral load resisting system provided that the stresses in these materials do not exceed (specified) values." However, there is concern that the contribution of these interior building partitions may be more significant than presently allowed by the new ordinance. Research that justifies increased allowable stresses for shear walls would be a significant factor in reducing the costs of strengthening when increased seismic resistance is required.

Independent of the development of the City Ordinance, and following a schedule that overlapped the adoption of the ordinance, a study was undertaken by Agbabian Associates under a grant from the National Science Foundation (Ref. 2) to investigate the effective participation of wood-framed interior shear wall partitions when determining the ultimate resistance capacity of two- and three-story masonry buildings to seismic loading. Wood-stud partition framing was stressed because (1) it is the type normally found in the older buildings that are especially susceptible to earthquake damage and (2) research directly applicable to this type of partition construction is very limited.

2. SCOPE OF RESEARCH PROGRAM

The study combined testing and analysis to investigate the in-plane shear load resistance characteristics of various combinations of lath and plaster materials commonly utilized for interior partition wall construction in pre-1934 unreinforced masonry buildings and assessed the influence of interior partitions on the safety of these buildings.

A series of static tests were conducted on four types of partition construction to investigate strength and rigidity to in-plane lateral forces, ultimate strength, and failure mode characteristics. The four types of partitions included:

Specimen A. Wood studs with 5/8-in. gypsum wallboard (horizontal joints)

Specimen B. Wood studs with 5/8-in. gypsum wallboard (vertical joints)

Specimen C. Wood studs with 3/8-in. gypsum lath and plaster

Specimen D. Wood studs with wood lath and plaster

The latter type was common in the construction of pre-1934 buildings. The test panels were analyzed using finite element techniques; predicted load vs. deflection relationships were correlated with test data to obtain material properties of strength and stiffness. The effectiveness of interior wall partitions was then assessed by including them in the analysis of typical masonry buildings with interior partitions of wood lath and plaster construction.

3. SUMMARY OF TESTS

Test panels were 4 ft high by 8 ft long constructed of 2 x 4 studs spaced at 16 in. on center (see Fig. 1a). Two panels were prepared for each partition type and bolted together, as shown in Figure 1b prior to applying the facing material. Details of the four types of facing material tested are shown in Figure 2.

The assemblies were then tested as simple beams, as shown in Figure 3. This method of testing results in an almost pure shear loading in each panel. Deflections were measured at the center and quarter points as loading was applied by a hand-pumped hydraulic jack. Care was exercised to ensure that load was transmitted to the wood framing members directly and not the facing material.

The progress of the cracks in the plaster was marked as the loading increased. A load vs. displacement curve is shown in Figure 4 for each specimen. Test results are summarized in Table 1. In Table 2, the shear per linear foot is given for each panel assemblage as measured at first cracking, at a noticeable break in load/deflection curve, and at failure. A comparison of test shear values with allowables in the City Ordinance show substantial factors of safety for wood studs with wood lath and plaster and reasonable factors of safety for wood studs with gypsum lath and plaster. Tests conducted to determine the lateral strength of nailing between wood lath and studs indicated that the lateral resistance of the nails is adequate to transfer the measured shear loads between the partition framing and wood lath and plaster facing material.

It must be noted that these tests were exploratory and that further studies are required to substantiate the preliminary conclusion that the resisting capacity of interior partitions is higher than what the City Ordinance acknowledges.

It should be noted that both the City Ordinance and the tests presume that the partitions are attached to joists or rafters in such a manner that the connection transfers the applied loads without failure.

4. PARAMETERS FOR ANALYSIS

The test panel assemblages were analyzed by using a finite element model using beam elements for studs and plane stress elements for wall facing materials. The material properties to represent each type of material are given in Table 3. A typical comparison of calculated and measured load/deflection curves is shown in Figure 5. It is noted that the analysis considers elastic properties. These analyses are exploratory, and further investigation is needed before generalizations can be made.

Finite element models may be considered appropriate in determining natural frequencies of interior panels and in dynamic analyses of the response of building/interior partition systems to seismic motions.

5. EFFECTIVENESS OF SHEAR WALL PARTITIONS

An evaluation was made of the effectiveness of wood-framed shear wall partitions in resisting the lateral loads imposed on two- or three-story masonry apartment/hotel buildings where extensive use has been made of floor-to-ceiling interior walls to partition off floor space. These structures normally use wood joists and sheathing for the floor and roof systems. Figure 6 shows the layout of apartments and arrangement of partitions assumed for a typical floor. For the purpose of the analyses, it was assumed that the exterior masonry walls and interior partitions are adequately connected to floor and roof diaphragms.

Calculations were made for the equivalent static case where the horizontal shear force is equal to 10% of gravity load. This corresponds to a medium risk building as defined in the City Ordinance. The maximum shear in transverse partitions was developed for two-story and three-story buildings based on tributary area assumptions. The shear loads were then compared with the shear wall values determined by tests of wood lath and plaster construction. The results are tabulated in Table 4.

The factors of safety for 10% of gravity indicate a reasonable margin against building failure. However, this observation may be premature since it is based on the results of only one test and depends on the factor of safety to be assigned for wood lath and plaster facing material.

6. RECOMMENDATIONS FOR FURTHER STUDY

Additional testing of wood lath and plaster partitions have been recommended. These tests would include racking tests of 8 ft by 8 ft panels using static cyclic loading to determine strength and stiffness characteristics and degradation due to shear stress and deformation under load reversal.

An investigation should also be made of construction practices that were used prior to the 1930's to attach partitions to floors, ceilings, and cross walls and of the manner in which seismic loads are transferred to lateral-resistance structural elements or systems through these connections. A program for testing existing connections should be included in the investigation. Methods for strengthening existing connections should also be developed and tested as part of this effort.

REFERENCES

1. *Los Angeles Municipal Code.* "Earthquake Hazard Reduction in Existing Buildings," Ordinance No. 154807, Approved Jan 7, 1981.

2. Anderson, R.W., *Investigation of the Seismic Resistance of Interior Building Partitions, Phase I*, R-8110-5205. El Segundo, CA: Agbabian Associates, Feb 1981.

TABLE 1. SUMMARY OF TEST RESULTS

Specimen	First Cracking		Effective Yield		Ultimate	
	Load, lb	Deflection, in.	Load, lb	Deflection, in.	Load, lb	Deflection, in.
A	1,800	0.27	2,300	0.5	3,800	1.8
B	2,250	0.20	4,000	0.4	6,300	1.1
C	4,050	0.12	6,000	0.25	9,225	1.1
D	8,300	0.19	10,000	0.25	14,900	0.65

TABLE 2. SHEAR WALL VALUES PER FOOT

Specimen	Test Results			Code Allowables	Factor of Safety		
	1st Crack	Yield	Ultimate		1st Crack	Yield	Ultimate
A	112	144	238	--	--	--	--
B	140	250	400	--	--	--	--
C	253	375	575	200	1.3	1.9	2.9
D	518	625	930	100	5.2	6.3	9.3

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TABLE 3. MATERIAL PROPERTIES FOR FINITE ELEMENT MODEL

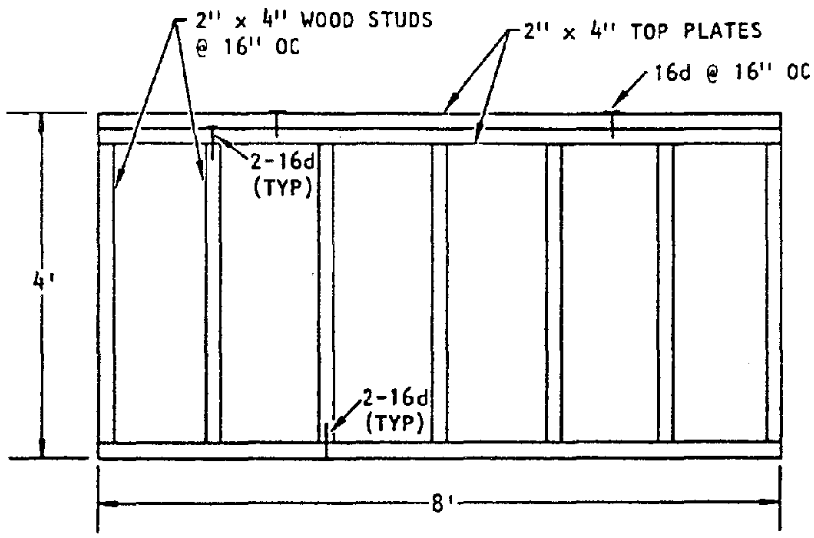
Material	Modulus of Elasticity E, psi	Shear Modulus G, psi	Poisson's Ratio, γ
Wood Stud	1.6×10^6		0.4
Gypsum Wallboard (Horizontal Joint)	2,400	1000	0.2
Gypsum Wallboard (Vertical Joint)	4,800	2000	0.2
*Gypsum Lath and Plaster	12,500	5200	0.2
*Wood Lath and Plaster	17,000	7100	0.2

*Combined Moduli

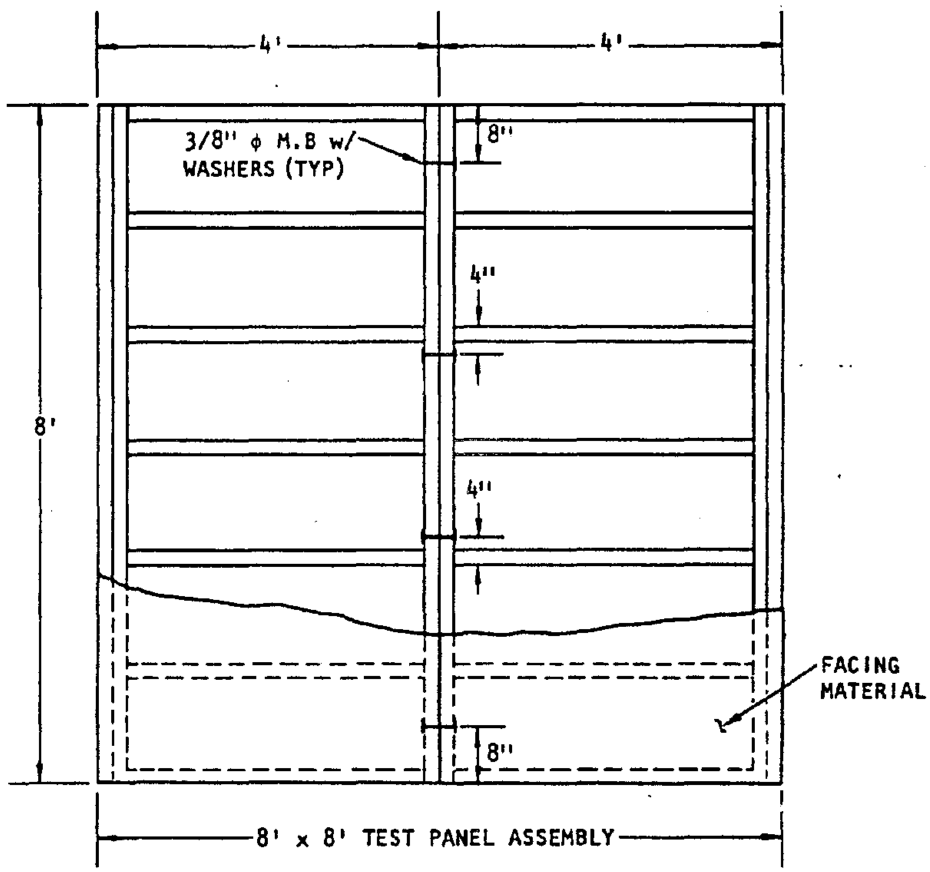
TABLE 4. COMPARISON OF TEST RESULTS WITH PARTITION SHEARS

Wood Lath and Plaster Test Panel Results lb/ft		Lateral Seismic Shear in Partitions							
		Three-Story Building				Two-Story Building			
		lb/ft (L/H ≧ 1.0)	FS	lb/ft (L/H not considered)	FS	lb/ft (L/H ≧ 1.0)	FS	lb/ft (L/H not considered)	FS
1st Cracking Shear	518	412	1.3	322	1.6	253	2.0	198	2.6
Effective Yield Shear	625	412	1.5	322	1.9	253	2.5	198	3.2
Ultimate Shear	930	412	2.3	322	2.9	253	3.7	198	4.7

AA10826

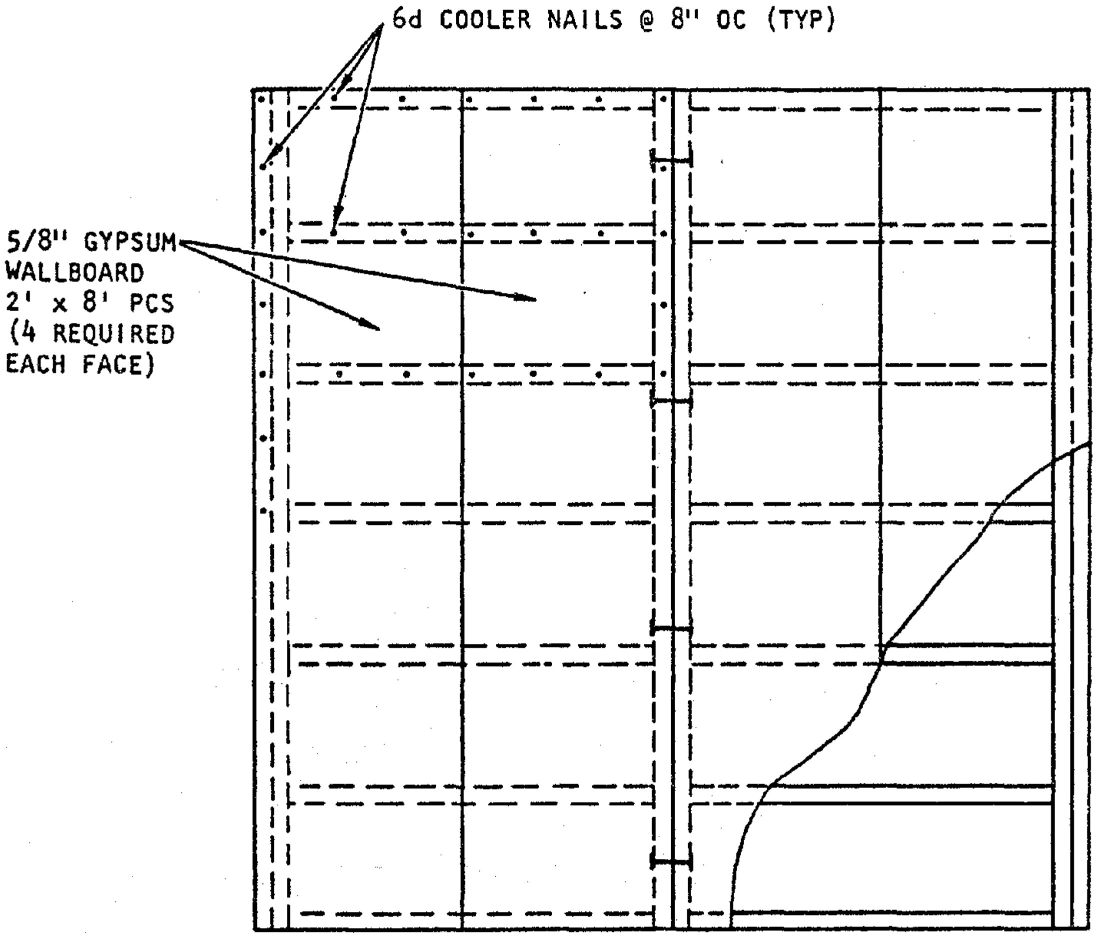


(a) Typical test panel framing



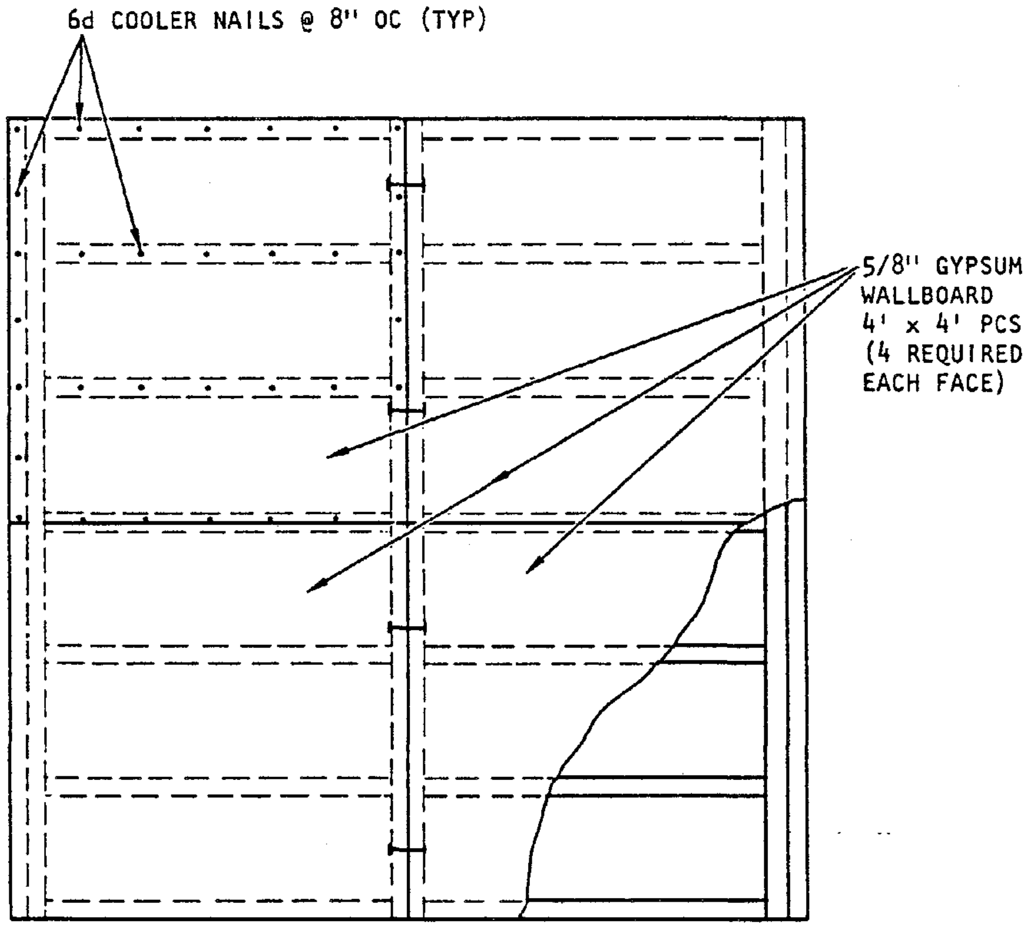
(b) Typical test panel assembly

FIGURE 1. TEST PANEL CONSTRUCTION



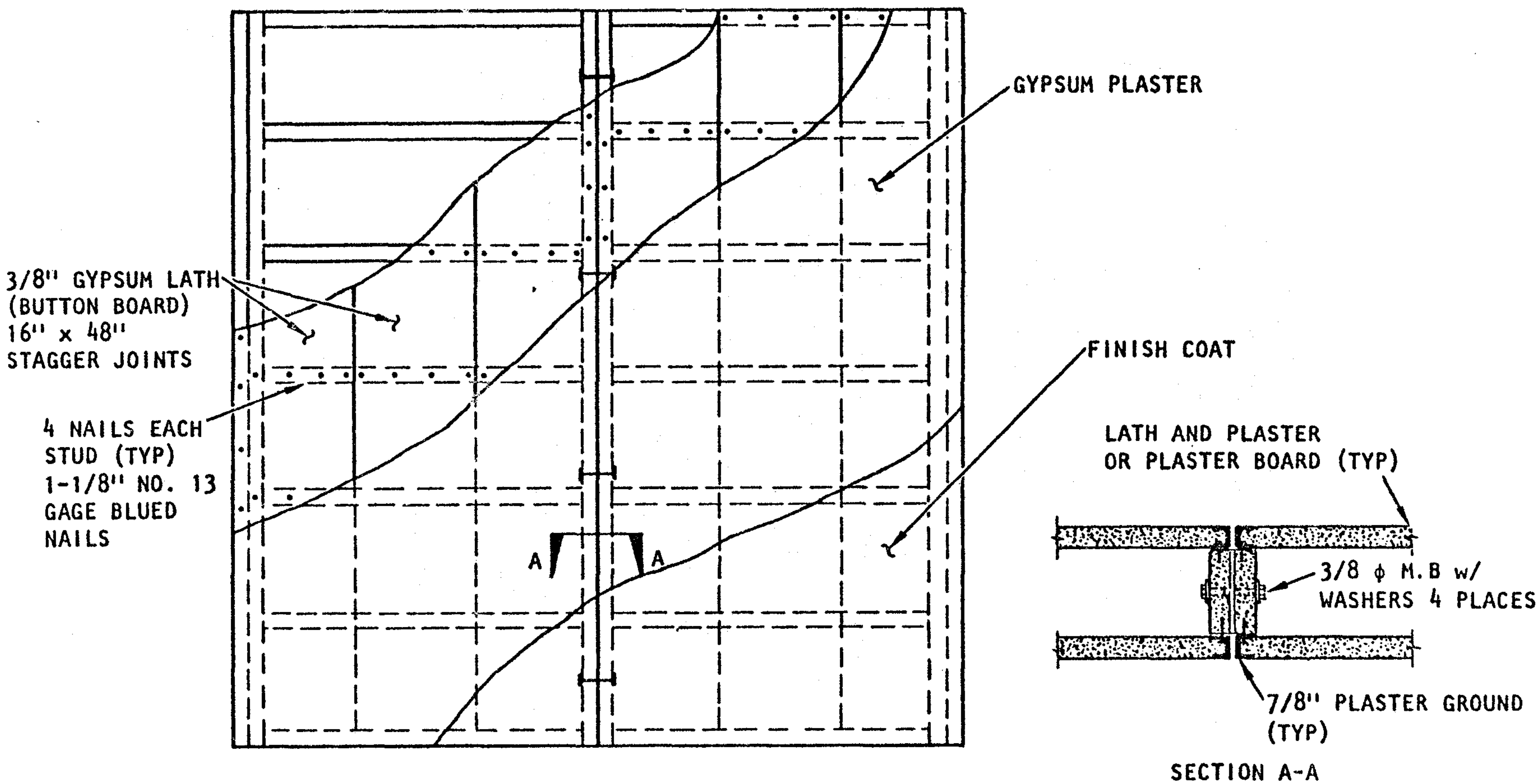
(a) Specimen A — gypsum wallboard (horizontal joint)

FIGURE 2. TEST PANEL DETAILS



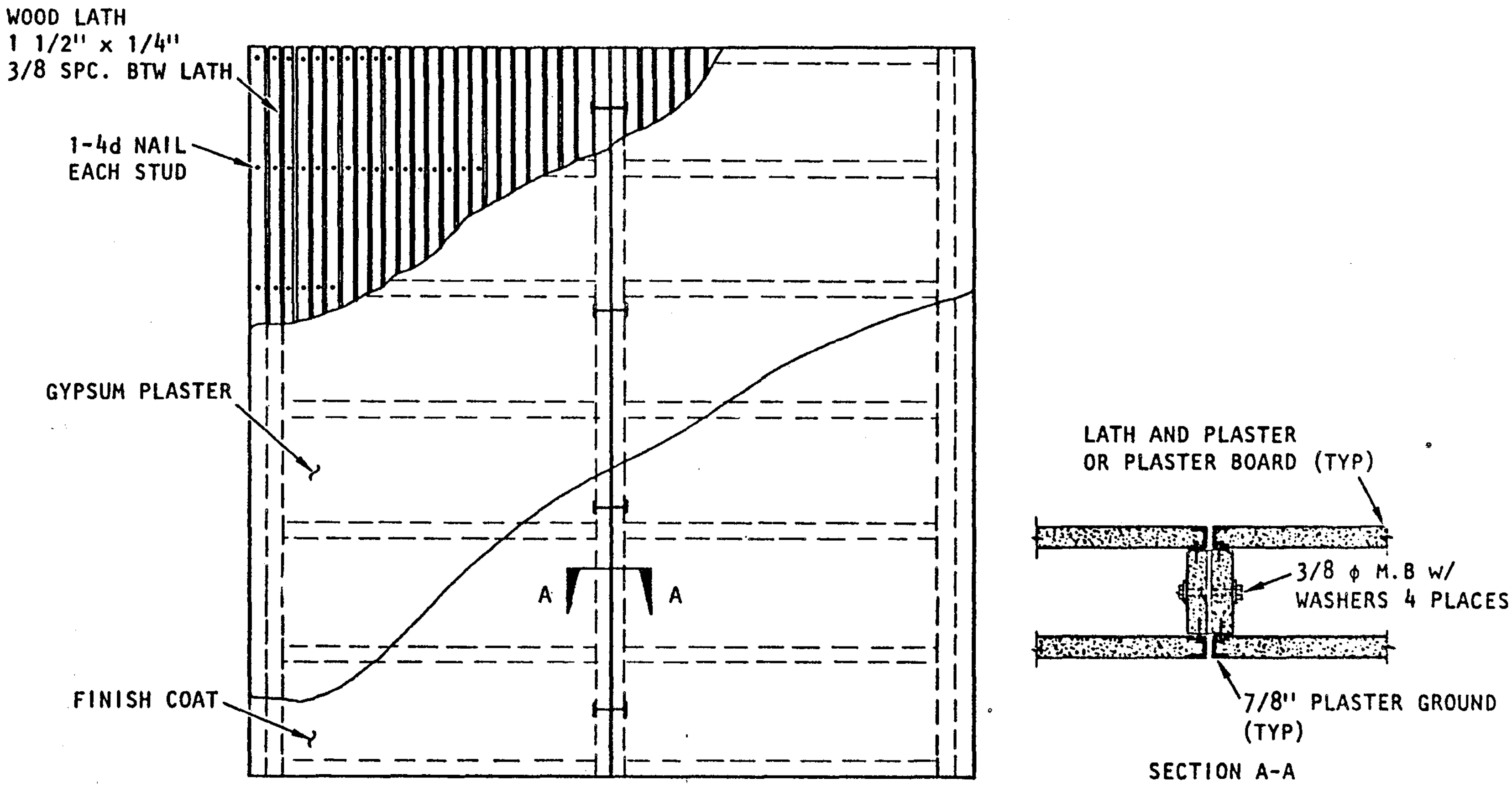
(b) Specimen B — gypsum wallboard (vertical joint)

FIGURE 2. (CONTINUED)



(c) Specimen C — gypsum lath and plaster

FIGURE 2. (CONTINUED)



(d) Specimen D – wood lath and plaster

FIGURE 2. (CONCLUDED)

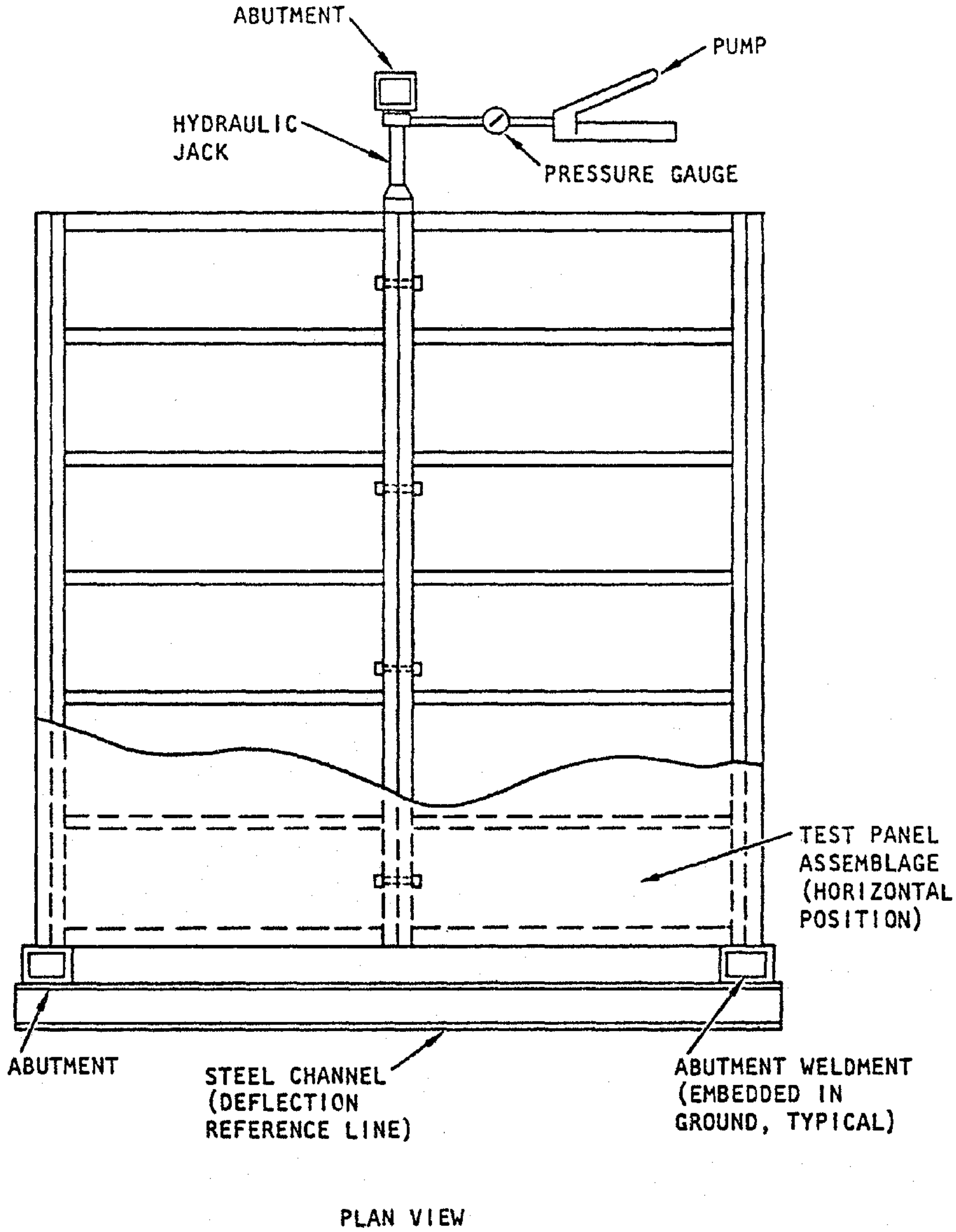


FIGURE 3. TEST SET-UP

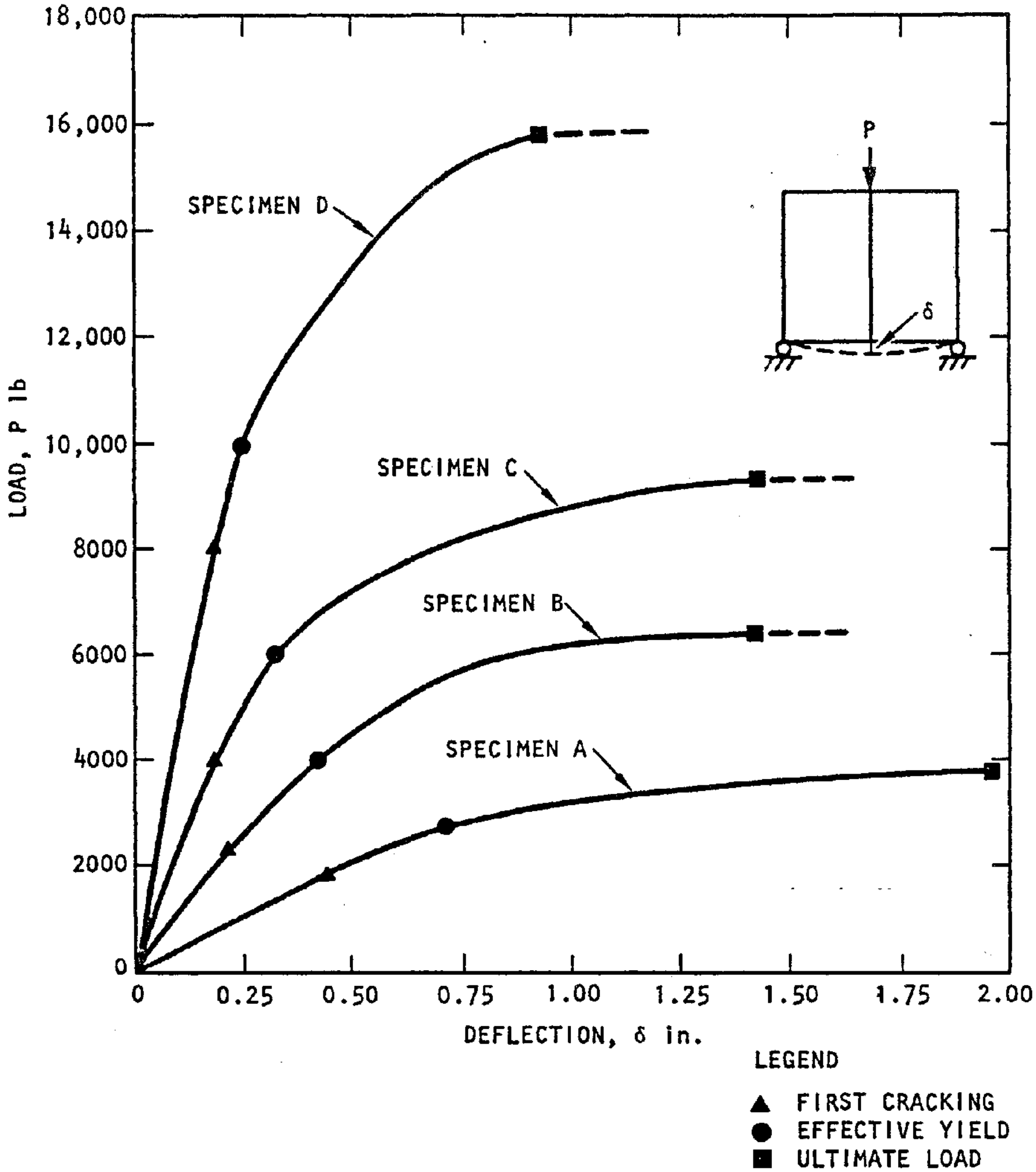


FIGURE 4. LOAD VS. DEFLECTION CURVES FOR EACH SPECIMEN

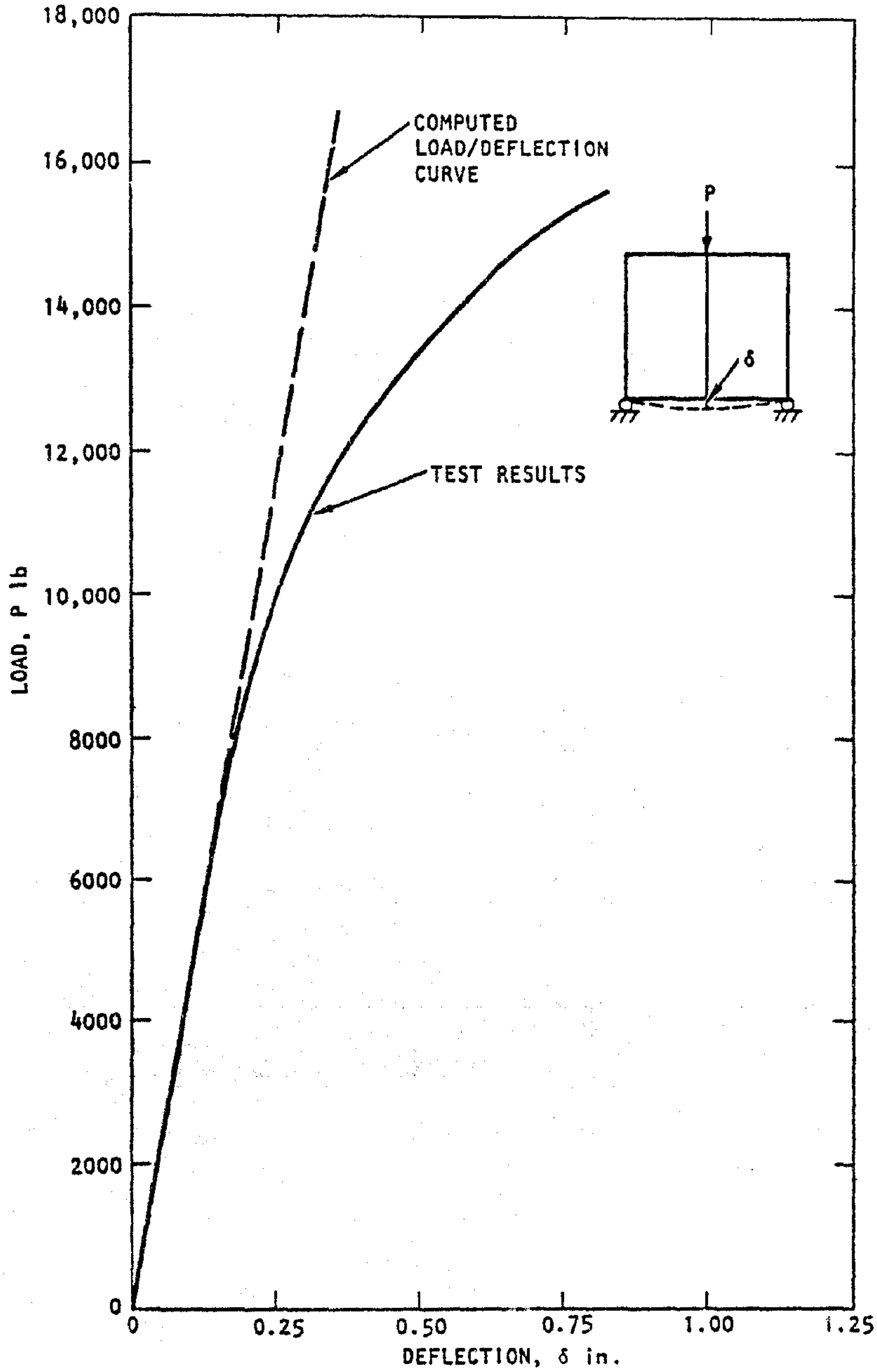
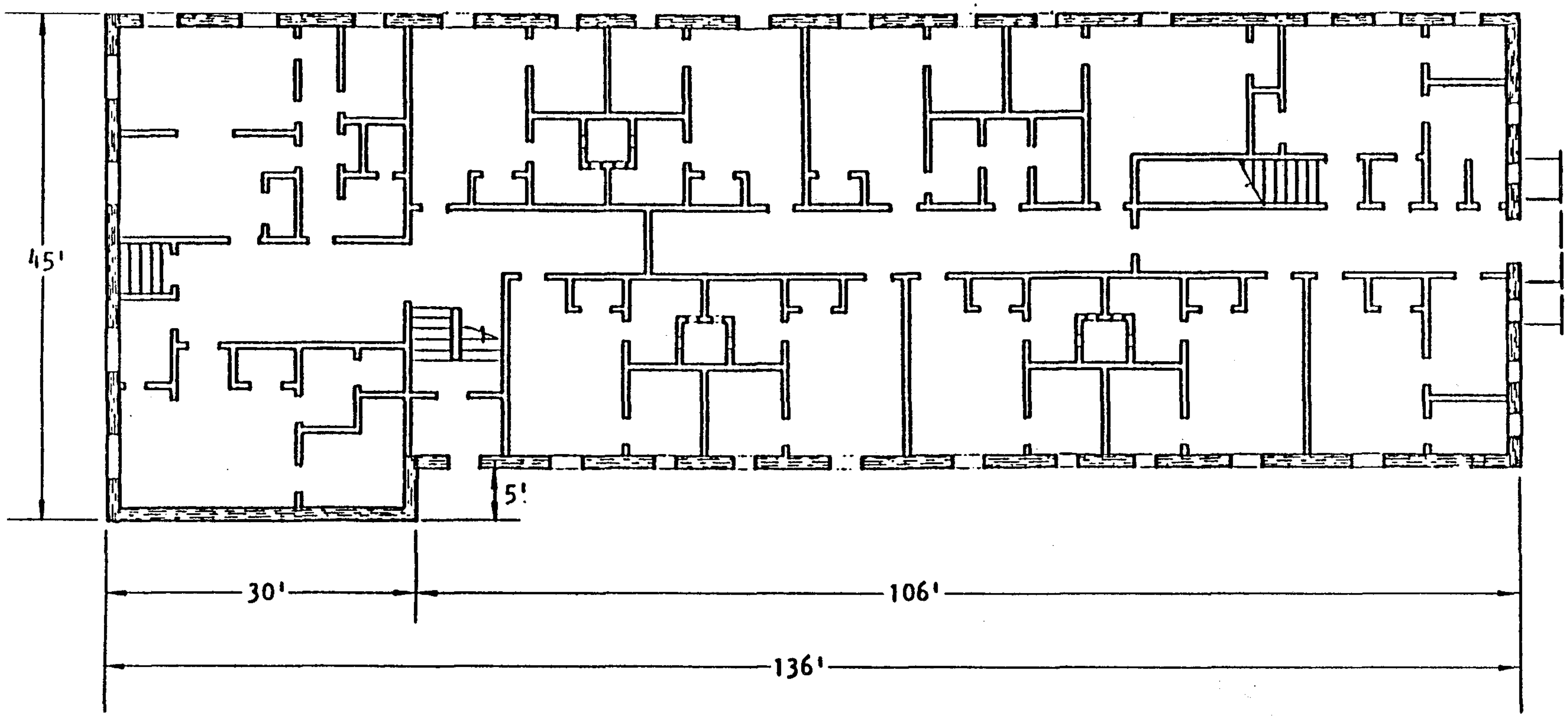


FIGURE 5. COMPARISON OF COMPUTED LOAD/DEFLECTION CURVE WITH TEST RESULTS — SPECIMEN D WOOD LATH AND PLASTER



LEGEND

MASONRY EXTERIOR WALLS

2 x 4 WOOD STUD PARTITIONS WITH WOOD LATH AND PLASTER

FIGURE 6. THREE-STORY UNREINFORCED MASONRY BEARING-WALL APARTMENT BUILDING (Schmid et al., 1978)

Experimental Studies on Retrofitting of Reinforced Concrete Structural Members

Y. Higashi*, T. Endo** and Y. Shimizu***

Introduction

In the earthquake countries, the reinforced concrete building must be provided adequate strength, ductility or both of these factors against large response earthquake force. Recently, some of the reinforced concrete buildings, especially those built more than 10 years ago (when Reinforced Concrete Structural Standard of Architecture Institute of Japan was revised) were evaluated as not always secure in shearing force of column at severe earthquake in Japan. Therefore, the reliable, easy and short-term strengthening methods for existing reinforced concrete buildings are necessary and the development of these methods is desired in our society.

Following two basic policies would be considered as strengthening the existing reinforced concrete buildings;

1) increasing the ultimate strength of the reinforced concrete building.
2) increasing the ductility of the reinforced concrete building in absorbing earthquake energy by plastic deformation.

In this paper, considering these policies, eleven types of strengthening method are adopted. The existing single-bay, single-story frames and single-bay, three-stories frames with poor web reinforcement columns are infilled with precast concrete panels, steel bracing, steel frame and so on, and the effects on them are investigated through the tests under static, lateral cyclic loading reversals. Then the behaviors of all specimens are analysed by using inelastic frame models.

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1. Test Specimen

Fourteen models of single-bay, single-story reinforced concrete frames (77 and 78 series/about one-third scale), and eight models of single-bay, three-stories reinforced mortar frames (79 series/about one-eighth scale), which had columns with relatively poor web reinforcement (reinforcement ratio of hoops; $P_W = 0.11\%$), are strengthened with the following methods;

1) Infilled reinforced concrete wall cast in place (77 - No. 2, 79 - No. 2)

2) Precast concrete wall panels filled in frame

   a) two precast concrete walls simulated as side walls (78 - No. 3, 79 - No. 3)
   b) two precast concrete walls with door open (78 - No. 4)
   c) three precast concrete walls (77 - No. 3)
   d) three precast concrete walls with cotter (77 - No. 4)
   e) four precast concrete walls (78 - No. 5, 79 - No. 4)
   f) four precast concrete slit walls (74 - No. 6, 79 - No. 5)

3) Reinforcement with steel

   a) steel bracing (78 - No. 7, 79 - No. 6)
   b) steel inside frame (78 - No. 8, 79 - No. 7)
   c) steel truss (78 - No. 9)

4) Web reinforcement in the column with steel plates (78 - No. 2)

Eleven types of strengthening are adopted, and twenty-two specimens including three pure frames and two monolithic walls are provided as shown in Table 1.

Three series (77, 78 and 79 series) of tests are done in 1977, 1978 and 1979. The details of the specimens are shown in Fig. 1. The details of the connection between wall panel and frame are shown in Fig. 2. The physical characteristics of materials used in the tests are shown in Table 2.

2. Test Procedure

To investigate the cyclic behavior of strengthened frames, each specimen is subjected to similar sequences of reversed cycle deflections as illustrated in Fig. 3. The axial load N is kept constant, which is

$N = 11.8 \times 10^4$ N (12 ton) ; 77 and 78 series

$N = 2.95 \times 10^4$ N (3 ton) ; 79 series

for each column (corresponding to $\sigma_o = 2.94\ MP_a$ (30 kg/cm^2) compressive stress).

Fig. 4 shows the complete test set-up for the specimen 77 - No. 3 and 79 - No. 1.

At the top of the ground floor, the horizontal deflection from the fixed base block are measured. The strains of the reinforcing steel bars and the widthes of the cracks are also measured.

3. Test Results

Initial stiffness, the loads and the deflections at the critical points (the yielding point, the maximum and ultimate point), and C_E factor (mentioned in the latter part of this paper) of each specimen are summarized in Table 3. Fig. 5 shows the load-displacement curves of the fourteen specimens. The crack patterns of all specimens at the ultimate state are shown in Fig. 6. The idealization of load-deflection relations at positive side loading and the schematic comparison of the effect of strengthening are illustrated in Fig. 7.

According to the Newmark's study, non-linear earthquake response analyses of structures shows that the required ductility factor (μ) of the elasto-plastic systems, whose yield shear factor is C_y, can be estimated from the elastic spectral response acceleration C_E, by the following equation [28].

$$C_E = C_y \cdot \sqrt{2\mu - 1}$$

This equation is based on the equal potential energy.

Considering possible plastic deformation together with strength, C_E factor defined in the following formula is adopted as an index of antiseismic capability, in order to estimate test results in this report.

$$C_E = \left(\frac{Qu}{2N}\right) \sqrt{2\mu - 1}$$

in which Qu and N denotes the ultimate shear strength and the axial load which is simulated gravity force respectively, and μ is the ductility factor obtained from the test.

4. Analysis

The behaviors of the all specimens are analyzed by using inelastic frame models. This analytical technique consists of the simplified routine method and is widely used to design of reinforced concrete buildings [35]. The outline and results of the analysis are as follows.

1) Analytical Model

Each model of these specimens is shown in Fig. 8. The columns and beams are considered as the rigidly jointed elements. The wall panel is idealized as compressive bracing or, compressive bracing plus tensile bracing. Then the both ends of these bracing are simulated as pin joints. Some joints are simulated those with springs, and the others are not. The steel truss is considered as the member of truss with springs, and the steel frame is considered as the members of columns and beams with springs.

2) Load-Deflection Relation of Members

The moment-rotation relation at the end of members in the frame and the stress-strain relation of the additional walls are assumed to be polylinear. The load-deflection relation of spring, and the axial force-strain relation of the steel bracing are assumed as bi-linear, which are shown in Fig. 9.

3) Frame Analysis

First, the stiffness matrix of an individual member is derived from basic principle of the direct stiffness matrix method [35]. And, the stiffness matrix of an assemblage is obtained from superimposing the stiffness matrices of the individual members. Having obtained the total structure stiffness matrix, the solution stems from a routine set of matrix calculation applied to the stiffness equation. The analyses are performed with load incremental method, and the concept of a stress matrix is used in calculating the internal forces.

4) Results of Analysis

The calculated load-deflection relation is compared with the experimental load-deflection hysteresis curves in Fig. 10. The experimental load-deflection hysteresis curves in Fig. 10 are the skeleton curves of the positive side on cyclic loading. The shear force to holizontal displacement curve obtained from the analysis agrees comparatively well with that from the test for every specimen, although this analytical technique consists of the simplified routine method.

5. Conclusion

Based on the experimental and analytical results on the strengthened reinforced concrete frame tests reported herein, several conclusions may be deduced as follows:

1) The specimens with precast concrete panels without opening or with reinforced concrete infilled cast-in-place wall showed high strength. The specimens strengthened by steel bracing and steel frame have large C_E factor as much as these precast concrete walls. On the other hand the specimens strengthened by adding two precast concrete walls with side

openings and those with steel truss have the smaller C_E factor.

2) In regard to the single-story specimens, many of them have a tendency to fail in shear, but the three-stories specimens have a tendency to fail in bending.

3) The non-linear analysis technique reported in this paper will be efficient for design of strengthening on actual buildings, although this technique consists of the simplified routine method.

Thus the methods of strengthening and the analytical technique reported in this paper will be useful to prevent or decrease the earthquake disaster of the existing buildings.

Table 1. Specimen

77 series	No. 1 - F1	Pure frame
	No. 2 - PW	Post casted shear wall
	No. 3 - C3	Adding 3 precast concrete walls
	No. 4 - C3C	Adding 3 precast concrete walls with cotter
78 series	No. 1 - F2	Pure frame
	No. 2 - SP	Column web reinforced with steel plates
	No. 3 - C2A	Adding 2 precast concrete walls simulated as side walls
	No. 4 - C2B	Adding 2 precast concrete walls with door opening
	No. 5 - C4	Adding 4 precast concrete walls
	No. 6 - C40	Adding 4 precast concrete slit walls
	No. 7 - SB	Adding steel bracing
	No. 8 - SF	Adding steel frame
	No. 9 - ST	Adding steel truss
	No.10 - FW	Monolithic wall with frame
79 series	No. 1 - 3F	Pure frame
	No. 2 - 3PW	Post casted shear wall
	No. 3 - 3C2A	Adding 2 precast concrete walls simulated as side walls
	No. 4 - 3C4	Adding 4 precast concrete walls
	No. 5 - 3C40	Adding 4 precast concrete slit walls
	No. 6 - 3SB	Adding steel bracing
	No. 7 - 3SF	Adding steel frame
	No. 8 - 3FW	Monolithic wall with frame

Table 2. Physical Properties of Materials

Concrete

Specimen			$_{c}\sigma_{B}$
frame	77 series	No. 1 ~ No. 4	17.3 (176)
	78 series	No. 1 ~ No. 7	20.3 (207)
		No. 8 ~ No. 10	20.8 (212)
77 - No. 2 (post casted wall)			21.5 (219)
precast concrete wall	77	No. 3 No. 4	23.7 (242)
	78	No. 3 ~ No, 6	22.4 (228)
Mortar			46.5 (474)

Reinforcing Steel Bar

Name		$_{s}\sigma_{y}$		$_{s}\sigma_{m}$	
77 series	D13	390	(3980)	581	(5920)
	D 6	336	(3430)	560	(5710)
	3.3 ϕ	228	(2320)	326	(3320)
78 series	D13	388	(3960)	577	(5880)
	D 6	369	(3760)	523	(5330)
	3.3 ϕ	533	(5430)	648	(6610)

Steel Shapes and Plates

Name	$_{s}\sigma_{y}$		$_{s}\sigma_{m}$	
L-50 x 50 x 4	413	(4210)	638	(6510)
H-125x125x6.5x9	313	(3190)	436	(4450)
PL - 9	315	(3210)	431	(4390)
PL - 1	283	(2890)	345	(3520)

$_{c}\sigma_{B}$; concrete compressive strength (MP_a)

$_{s}\sigma_{y}$; steel yielding strength (MP_a)

$_{s}\sigma_{m}$; steel maximum strength (MP_a)

( ) ; CGS unit system (kgf/cm^2)

Table 2-b

Mortar	$_c\sigma_B$
Specimen	14.1 (144)
Fill-in mortal	24.1 (246)

Steel Bar	$_s\sigma_y$	$_s\sigma_m$
2ϕ	247 (2520)	330 (3370)
4ϕ	410 (4180)	476 (4850)
D6	363 (3700)	544 (5550)
D10	378 (3850)	552 (5630)

Steel	$_s\sigma_y$	$_s\sigma_m$
H -6	348 (3550)	496 (5060)
H -19	363 (3700)	491 (5010)
L - 30 x 30 x 3	399 (4070)	529 (5390)

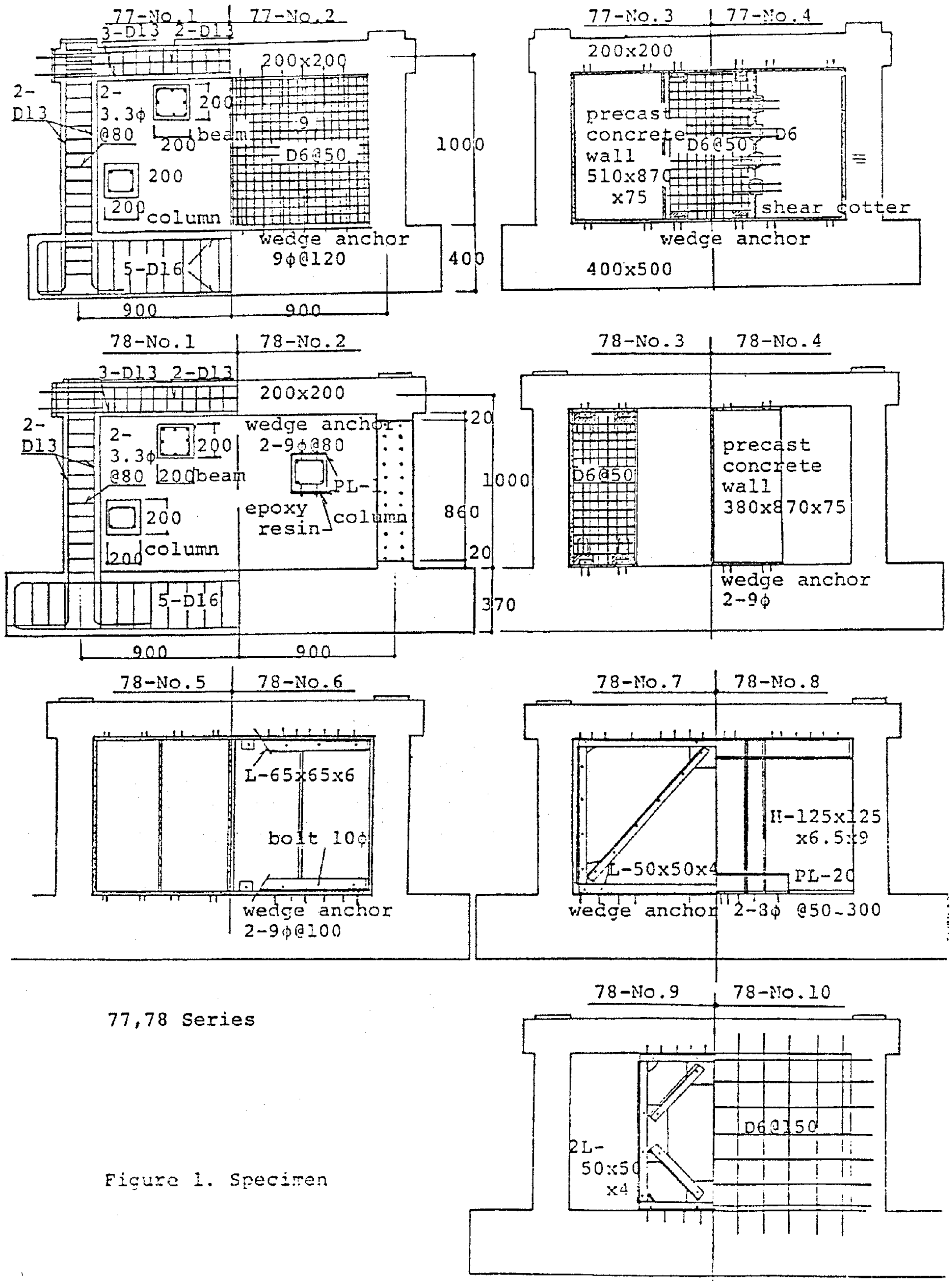
Table 3. Test Results

SPECIMEN		77 series				78 series									
		No. 1	No. 2	No. 3	No. 4	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6	No. 7	No. 8	No. 9	No.10
INITIAL STIFFNESS $\times 10^5$ N/cm ($\frac{ton}{cm}$)		2.18 (22.2)	44.6 (445.0)	13.3 (136.0)	18.8 (192.0)	2.53 (25.8)	2.99 (30.5)	13.9 (141.3)	7.82 (79.7)	48.3 (492.5)	14.1 (144.1)	7,27 (74.1)	4.29 (43.7)	3.68 (37.5)	58.8 (600.0)
YIELDING POINT	load $\times 10^4$ N Q_y (ton)	9.4 (9.6)	39.2 (40.0)	19.6 (20.0)	43.1 (44.0)	9.1 (9.3)	9.8 (10.0)	12.6 (12.8)	9.3 (9.5)	39.0 (39.8)	9.6 (9.8)	17.3 (17.6)	16.1 (16.4)	14.5 (14.8)	49.0 (50.0)
	deflection δ_y (cm)	1.15	0.47	0.45	0.86	0.96	1.00	0.50	0.50	0.50	0.20	0.48	0.76	0.77	0.51
MAXIMUM	load $\times 10^4$ N Q_{max} (ton)	10.5 (10.7)	39.2 (40.0)	32.4 (33.0)	45.1 (46.0)	10.9 (11.1)	11.5 (11.7)	15.4 (15.7)	14.2 (14.5)	39.2 (40.0)	15.7 (16.0)	25.6 (26.1)	25.7 (26.2)	18.2 (18.6)	56.9 (58.0)
	deflection δ_m (cm)	1.65	0.47	1.89	1.03	1.95	2.00	2.00	2.00	0.73	2.00	3.54	4.00	1.97	0.87
ULTIMATE	load $\times 10^4$ N Q_u (ton)	10.5 (10.7)	35.3 (36.0)	32.4 (33.0)	45.1 (46.0)	10.9 (11.1)	11.4 (11.6)	13.8 (14.1)	12.9 (13.2)	39.2 (40.0)	15.1 (15.4)	25.6 (26.1)	24.9 (25.4)	18.2 (18.6)	56.9 (58.0)
	deflection δ_u (cm)	1.65	0.73	1.89	1.03	1.95	3.52	3.35	2.56	0.73	4.00	3.54	5.50	1.97	0.87
C_E		0.61	2.18	3.74	2.26	0.81	1.19	2.07	1.67	2.31	4.01	4.03	3.88	1.57	3.75

(   ) ; CGS unit system (ton)

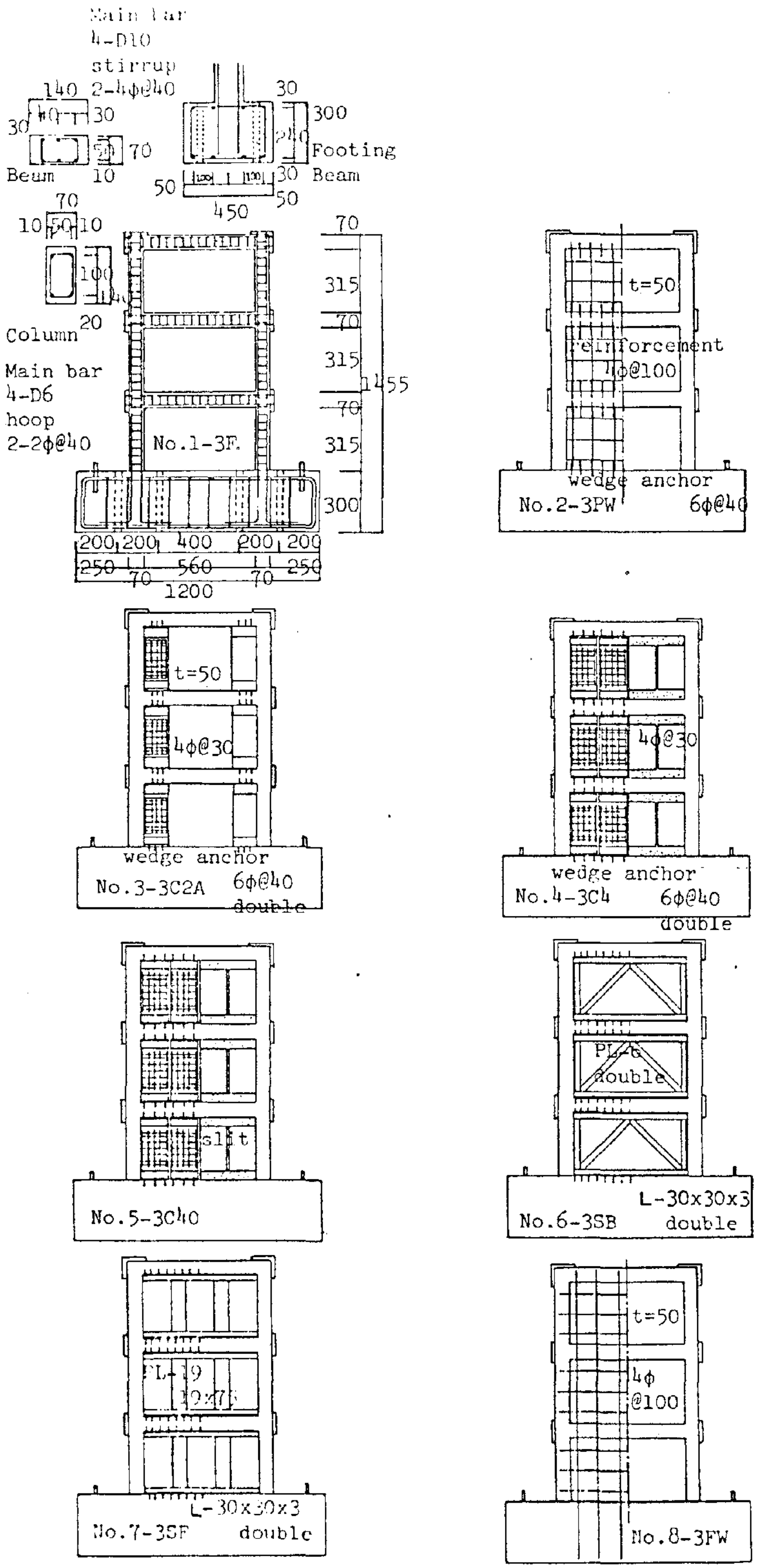
Table 3-b Test Results (at ground floor)

Specimen		79 Series							
		No. 1	No. 2	No. 3	No. 4	No. 5	No. 6	No. 7	No. 8
Initial Stiffness $\times 10^5$ N/cm (ton/cm)		0.59 (6)	14.71 (150)	1.27 (13)	4.41 (45)	5.39 (55)	4.41 (45)	3.43 (35)	15.20 (155)
YIELDING POINT	load Q_y $\times 10^4$ N (ton)	1.03 (1.05)	7.44 (7.59)	1.44 (1.47)	4.18 (4.26)	3.38 (3.45)	6.03 (6.15)	4.71 (4.80)	7.44 (7.59)
	deflection δ_y mm	2.2	2.3	2.1	5.2	2.0	4.4	4.4	2.9
MAXIMUM	load Q_{max} $\times 10^4$ N (ton)	1.91 (1.95)	8.47 (8.64)	2.97 (3.03)	5.74 (5.85)	5.12 (5.22)	7.80 (7.95)	5.97 (6.09)	8.47 (8.64)
	deflection δ_m mm	9.4	7.0	16.8	21.2	14.9	12.0	18.8	7.0
ULTIMATE	load Q_u $\times 10^4$ N (ton)	1.91 (1.95)	8.15 (8.31)	--	5.74 (5.85)	5.00 (5.10)	7.65 (7.80)	--	8.12 (8.28)
	deflection δ_u mm	9.4	9.8	20.0	21.2	17.8	15.1	20.0	9.0
C_E		0.89	3.80	2.15	2.61	3.48	3.15	2.89	3.15



77,78 Series

Figure 1. Specimen


Main bar
4-D10
stirrup
2-4ϕ@40
140
30
Beam
10
70
300
240
Footing Beam
450
50
10
Column
Main bar
4-D6
hoop
2-2ϕ@40
No.1-3F
315
1455
200 200 400 200 200
250 560 250
1200
t=50
reinforcement
4ϕ@100
wedge anchor
No.2-3PW
6ϕ@40
4ϕ@30
No.3-3C2A
double
No.4-3C4
slit
No.5-3C40
PL-6
double
L-30x30x3
No.6-3SB
No.7-3SF
No.8-3FW

79 Series

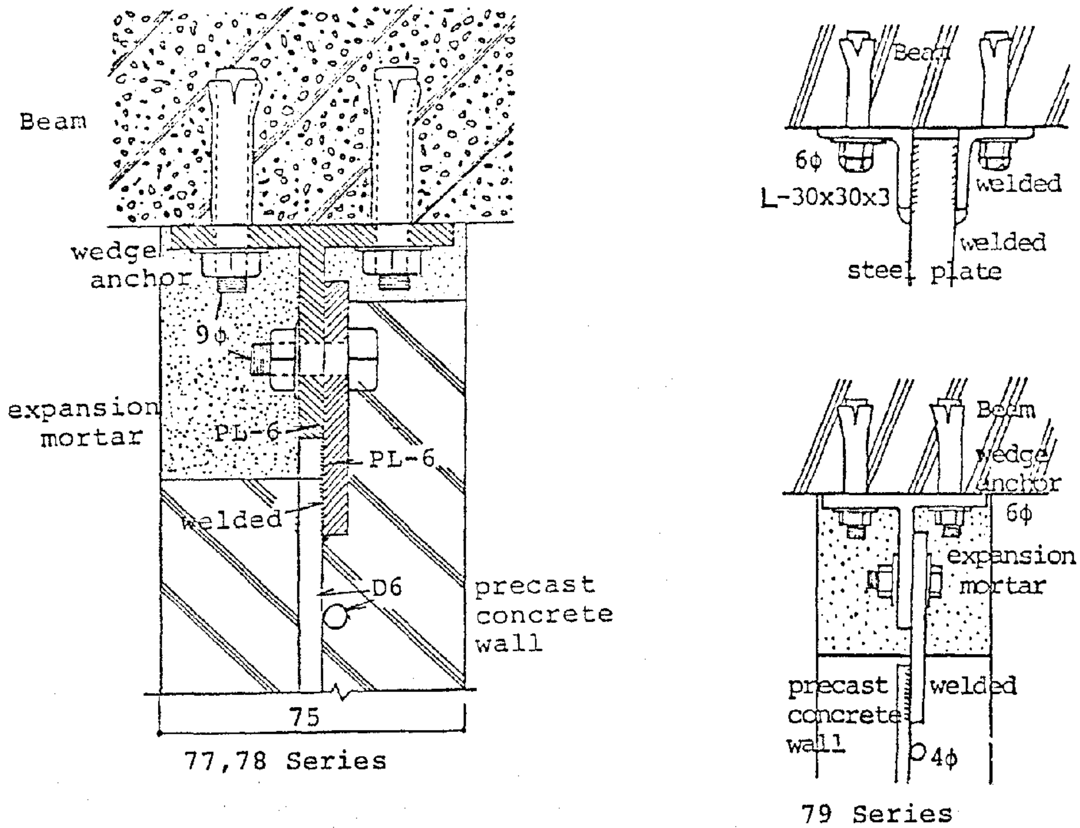


Figure 2. Detailes of the Connection between wall panel and frame

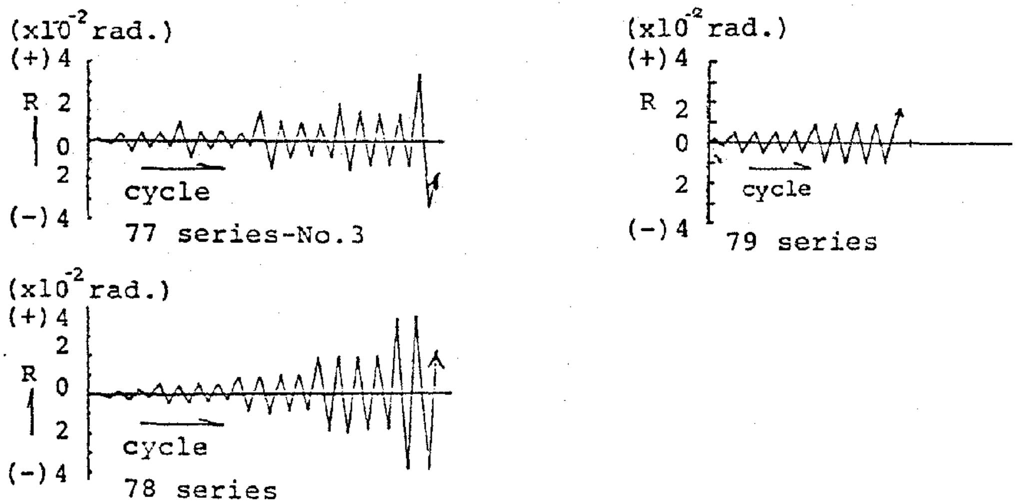
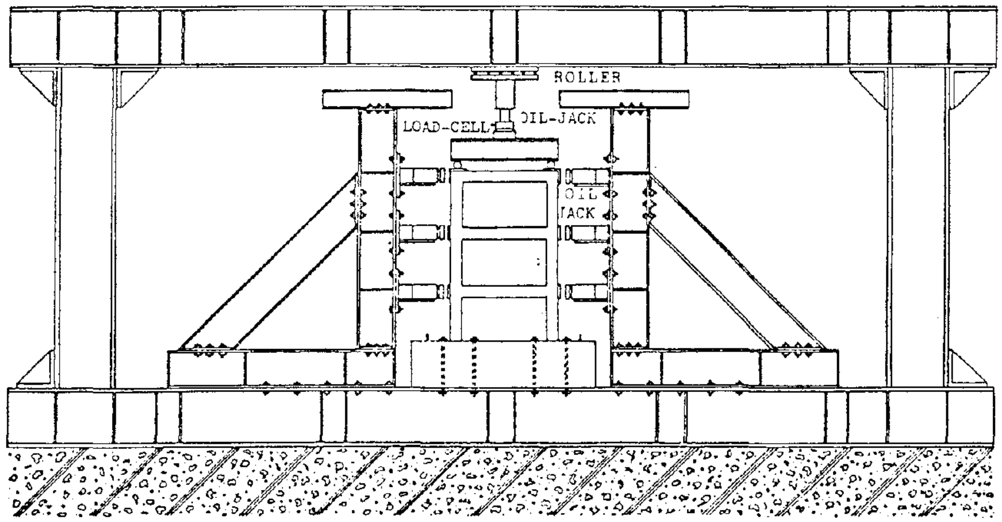


Figure 3.
Reversed Cycle Deflection
Sequence



Test set-up for specimen 79-No.1

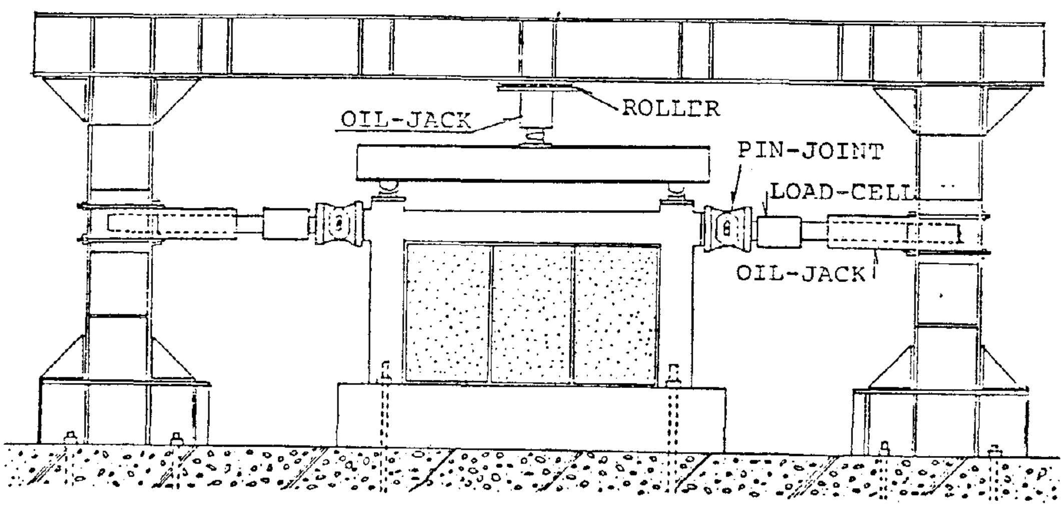


Figure 4. Test Set-up for Specimen 77-No.3

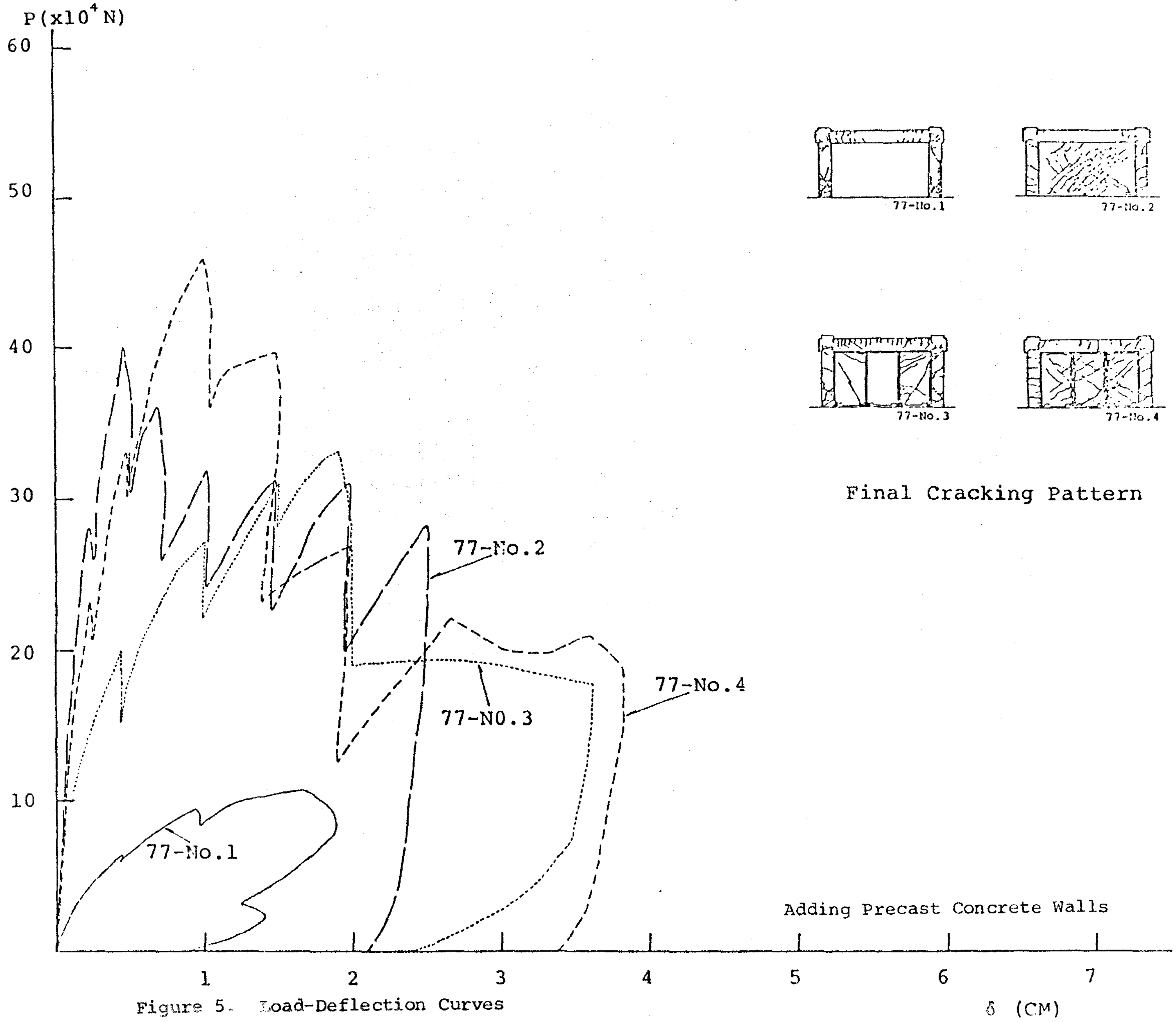
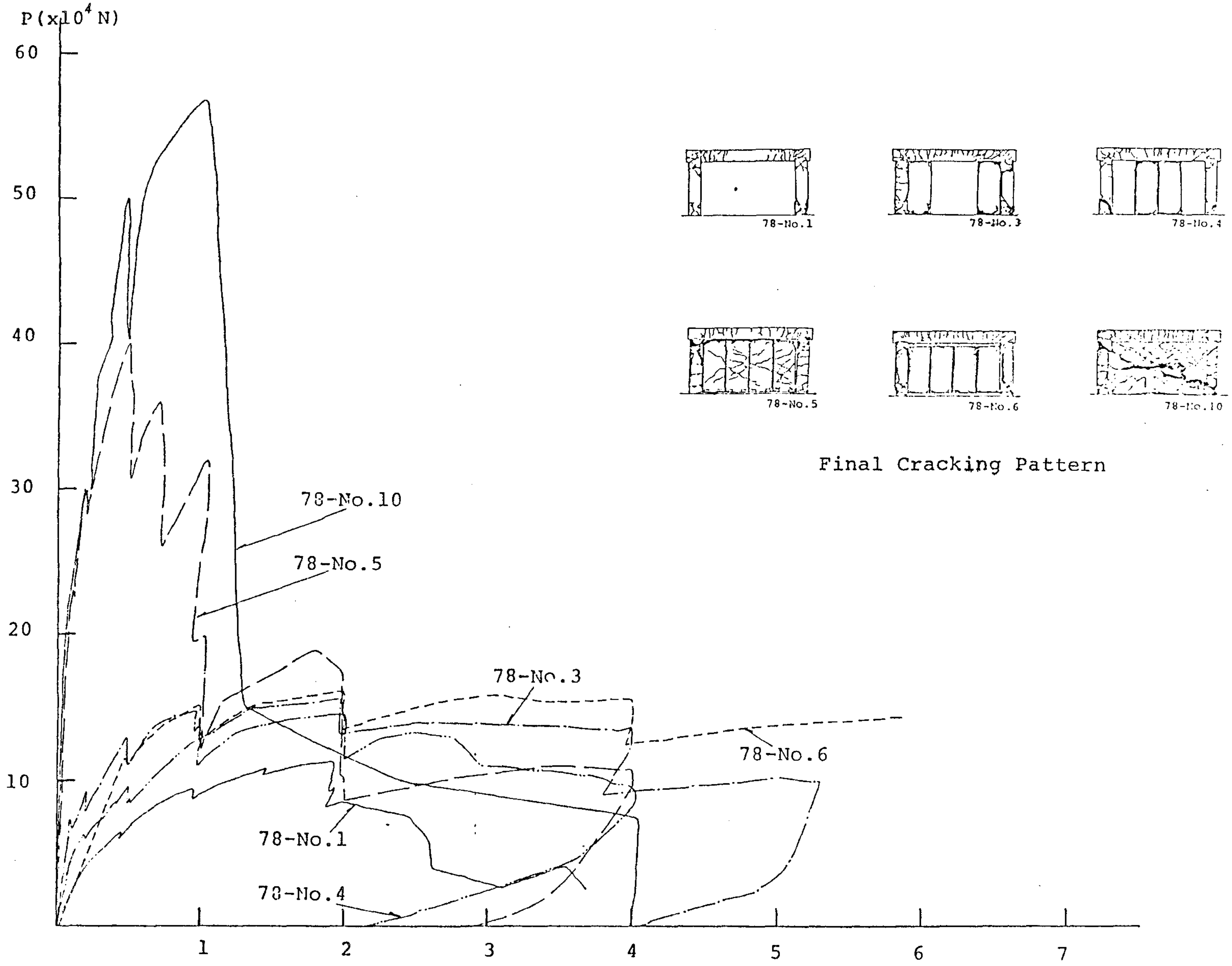
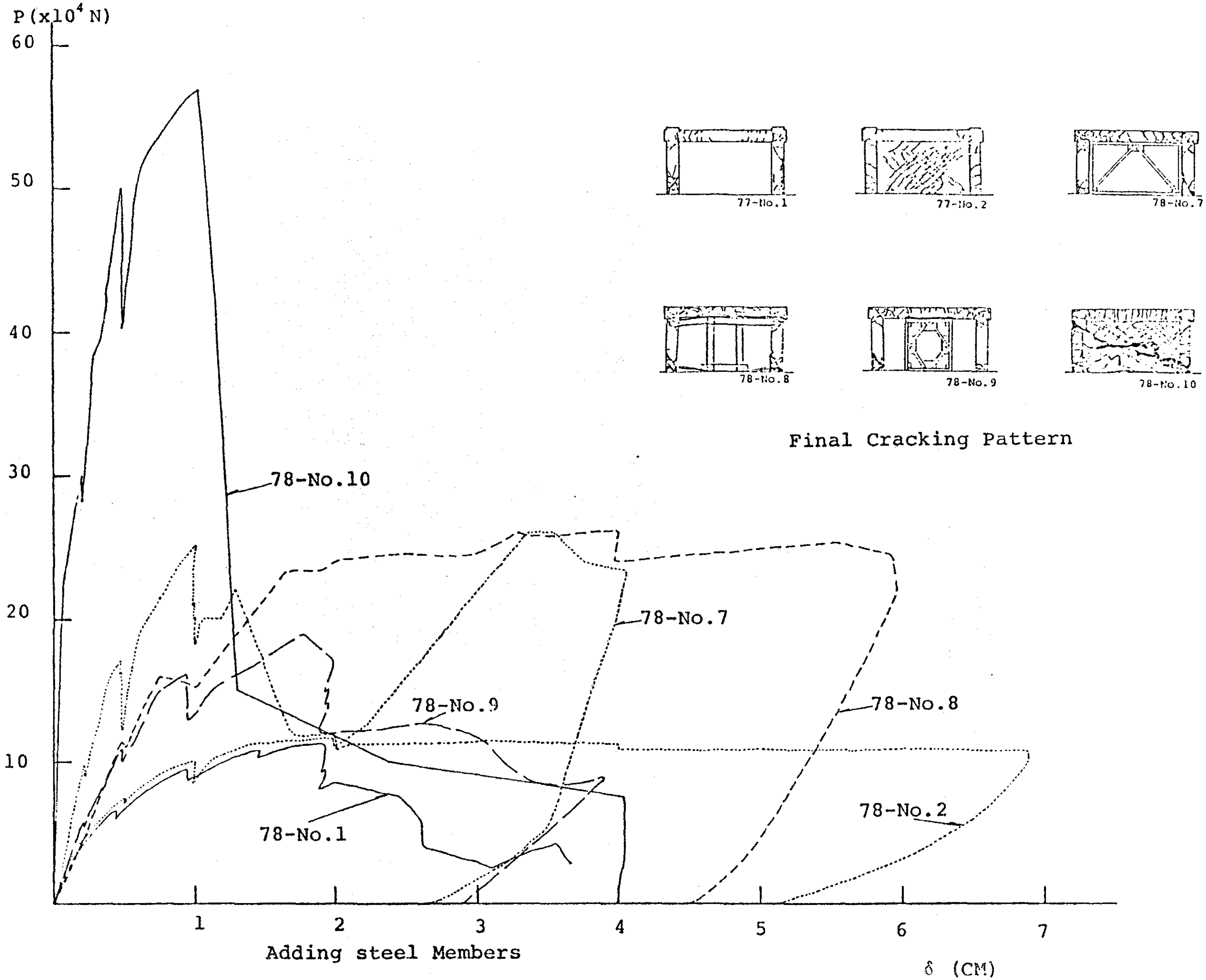
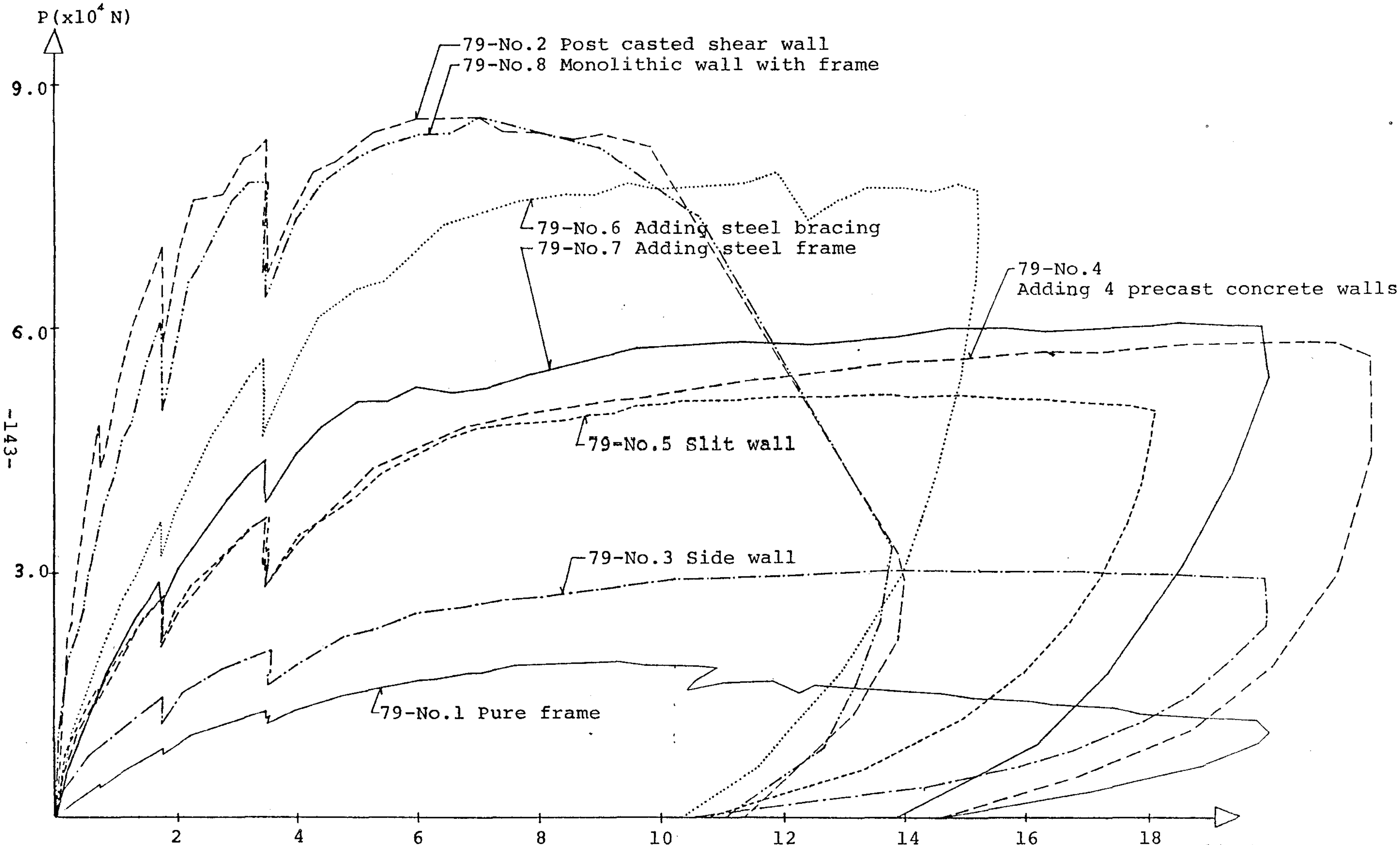


Figure 5. Load-Deflection Curves



Adding precast Concrete Walls


P (x10^4 N)
60
50
40
30
20
10
1
2
3
4
5
6
7
δ (CM)
Adding steel Members
78-No.10
78-No.7
78-No.9
78-No.8
78-No.1
78-No.2
77-No.1
77-No.2
78-No.7
78-No.8
78-No.9
78-No.10
Final Cracking Pattern



Single-bay, Three-stories Specimens

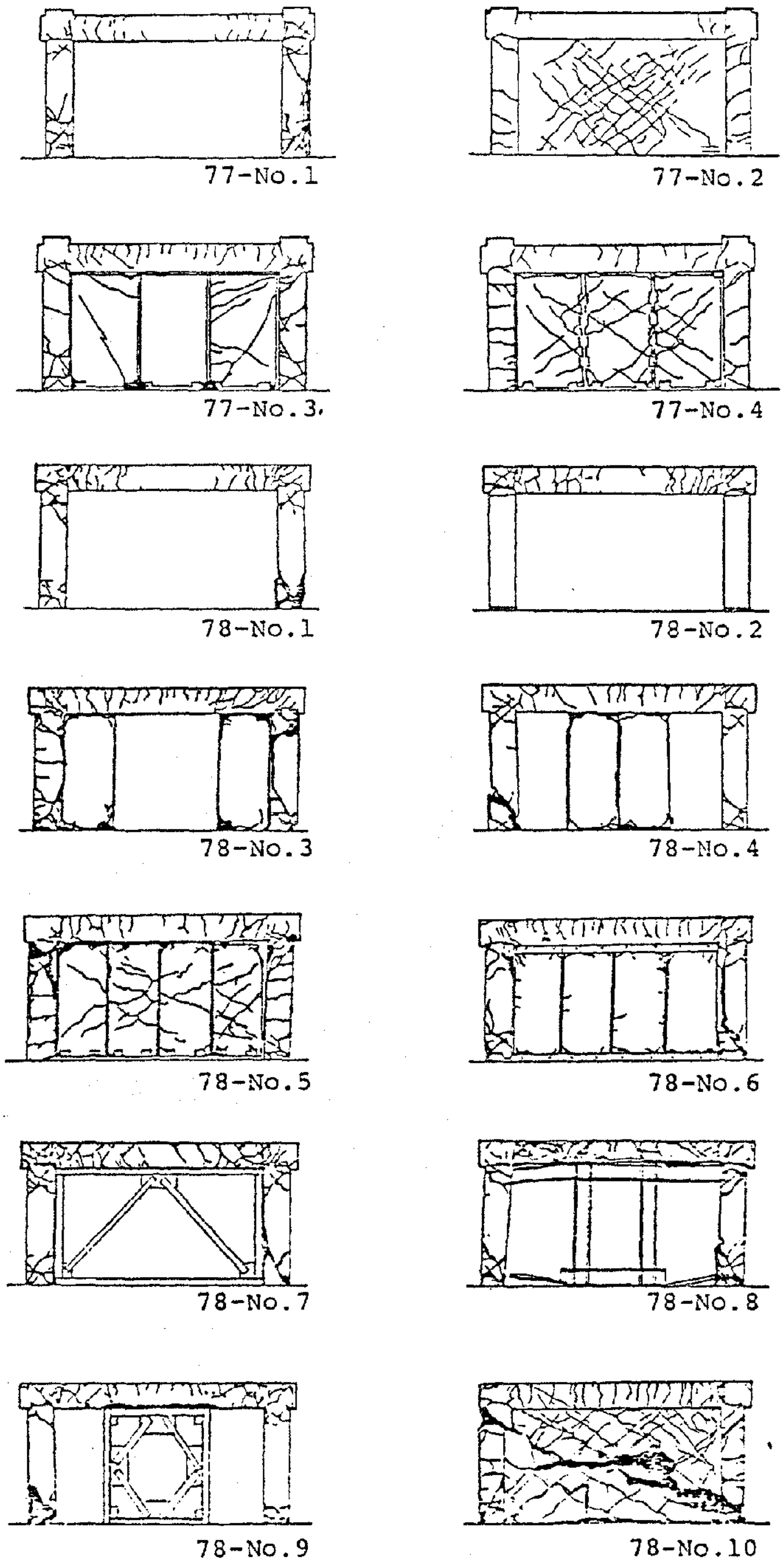
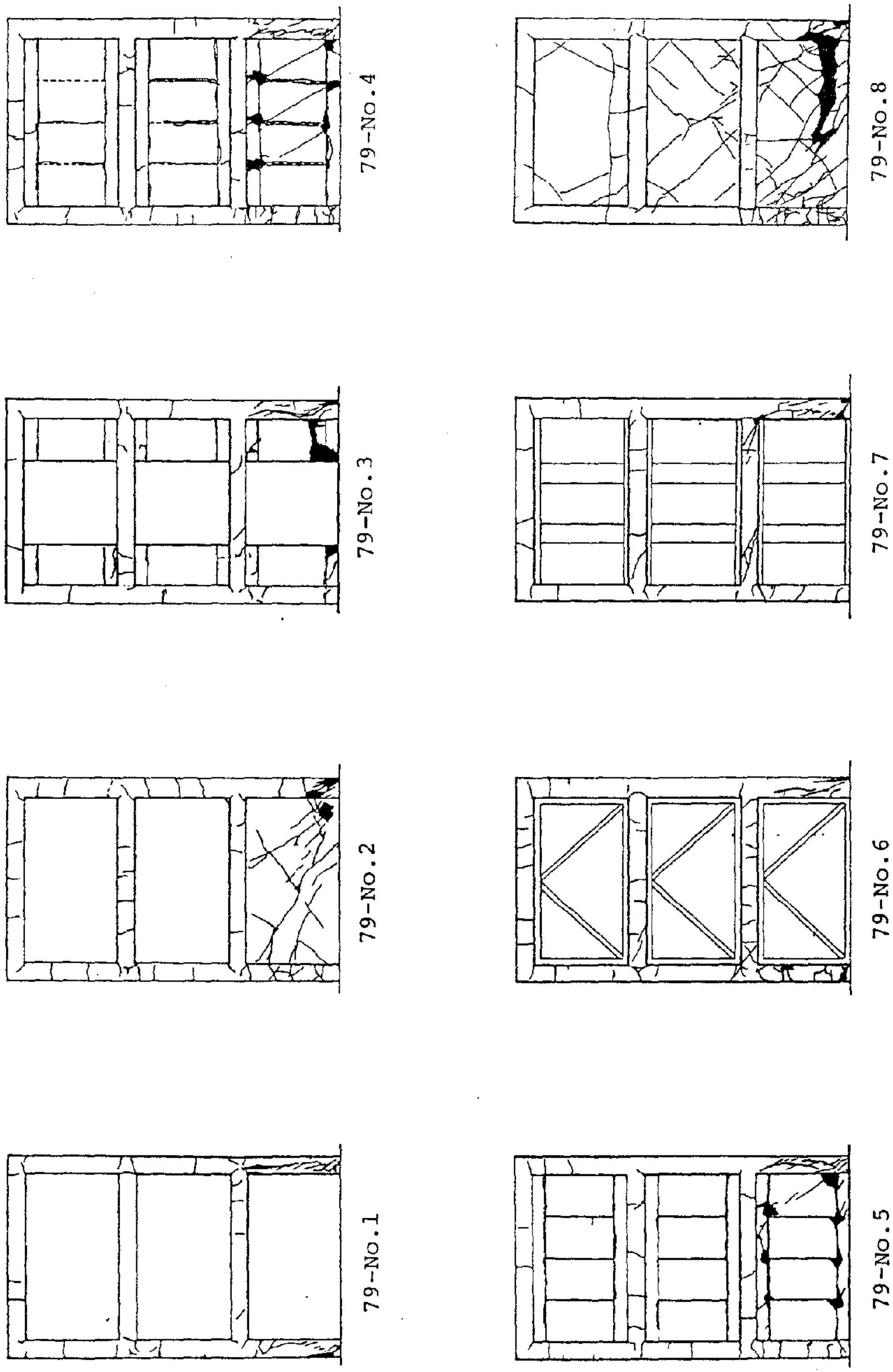
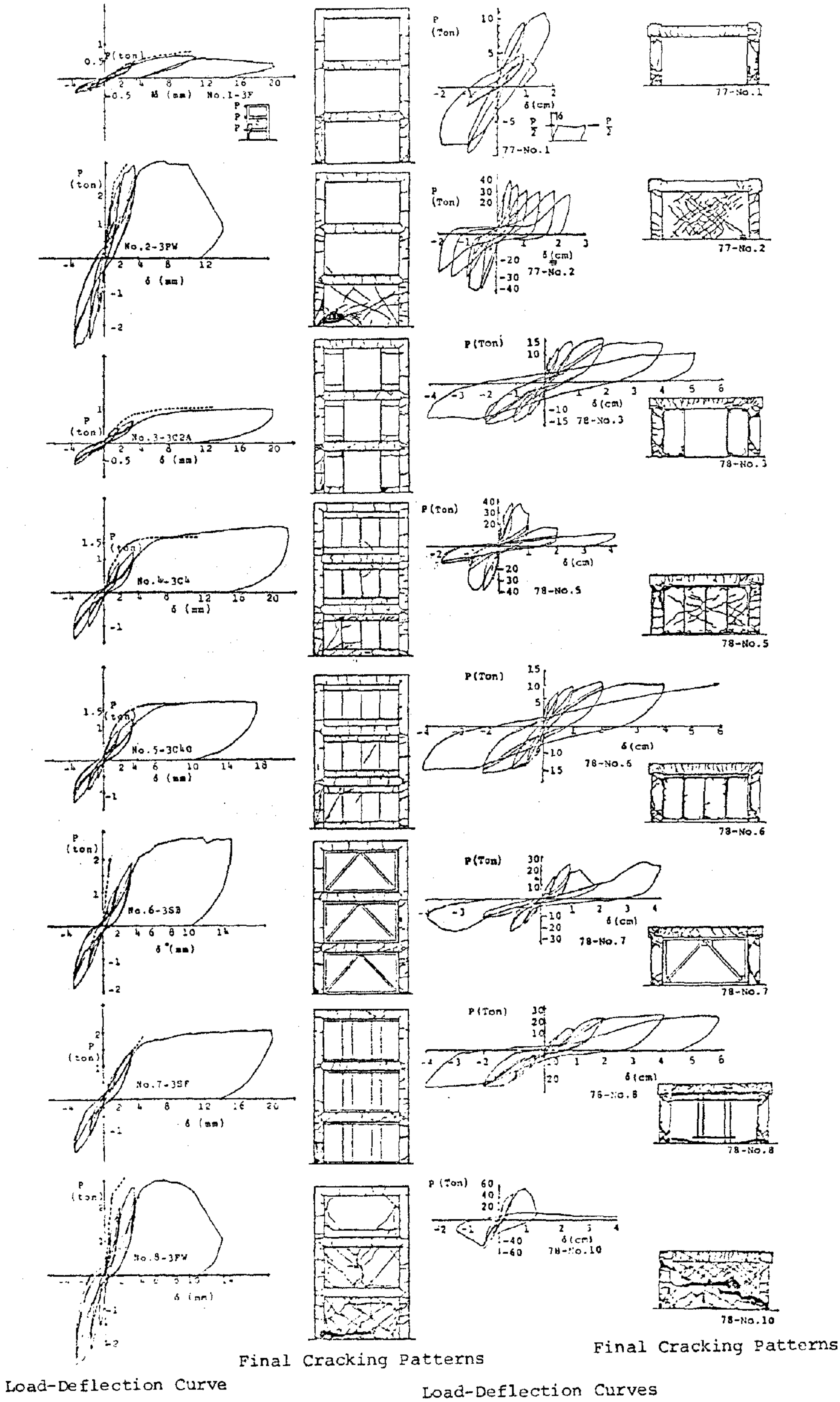


Figure 6. Final Cracking Pattern



79 Series



Comparison of the three-stories specimens with single-story specimens

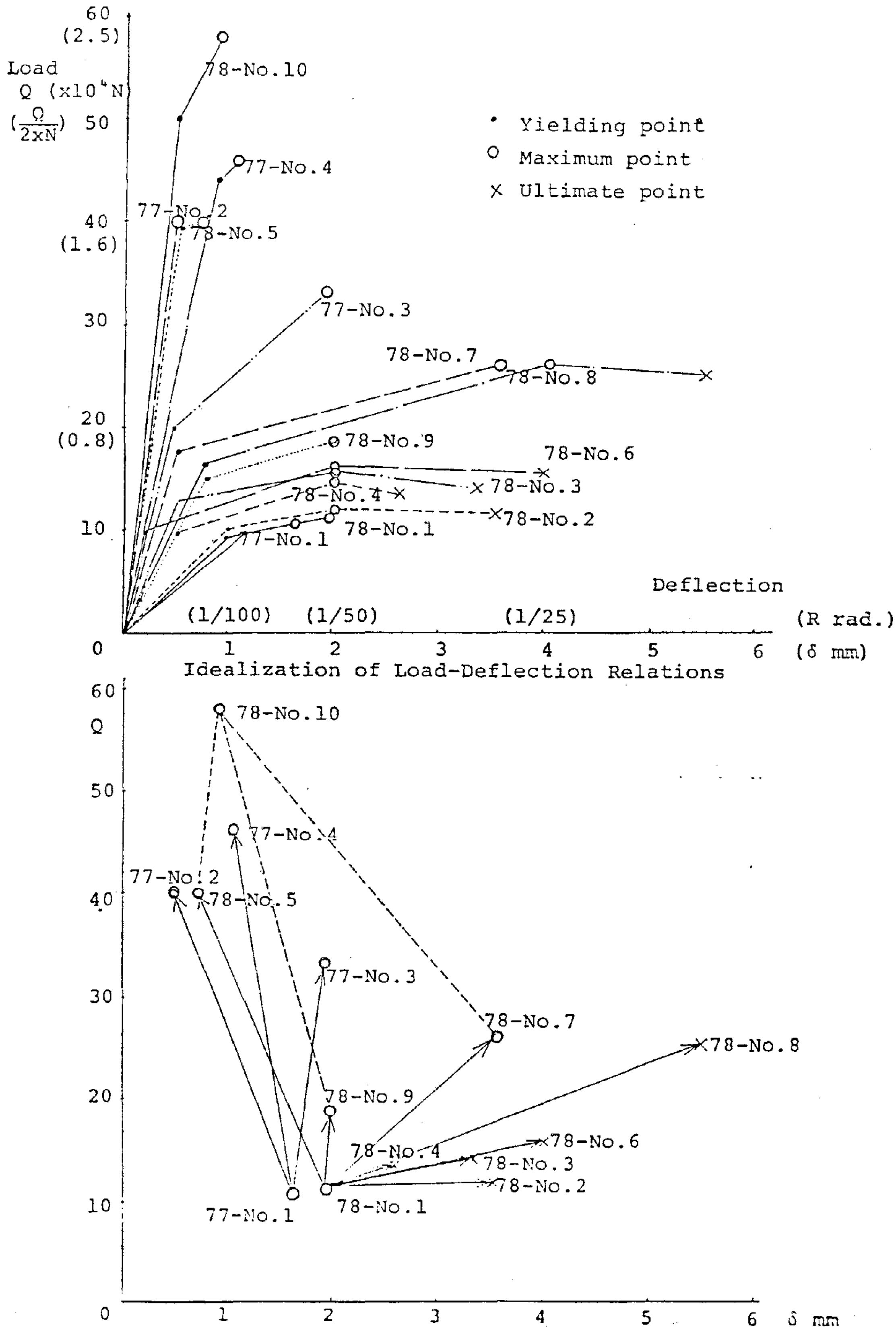
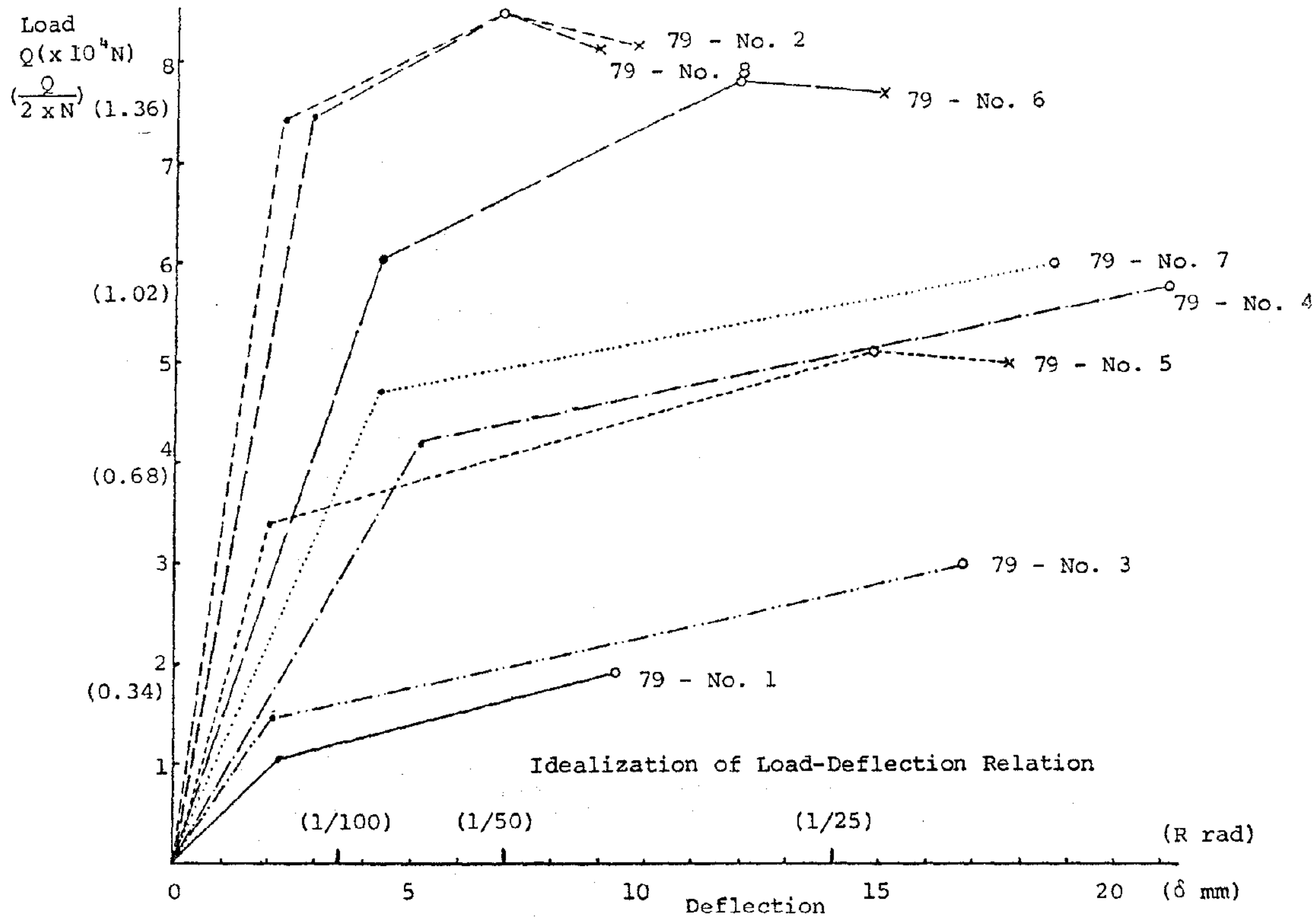
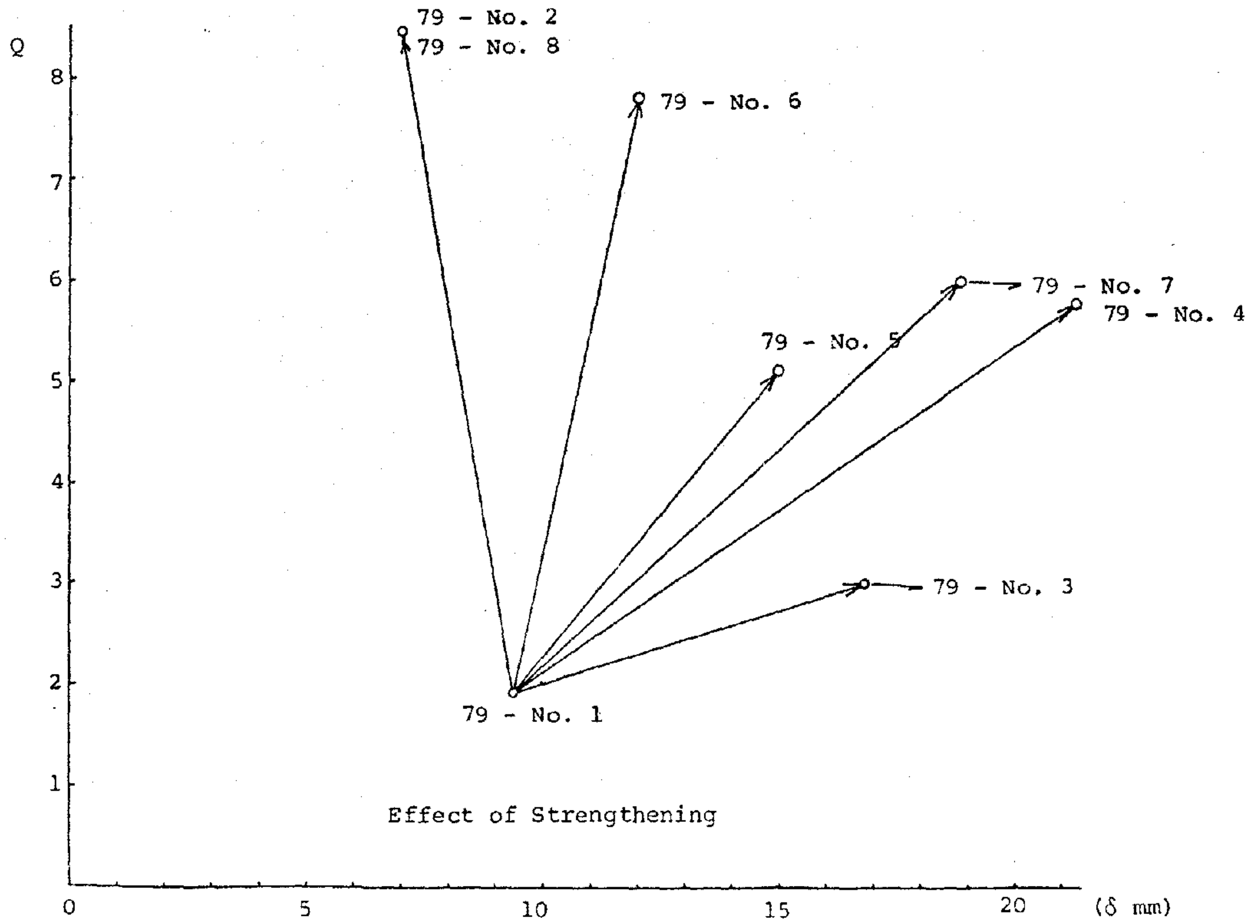


Figure 7. Effect of Strengthening



Idealization of Load-Deflection Relation



Effect of Strengthening

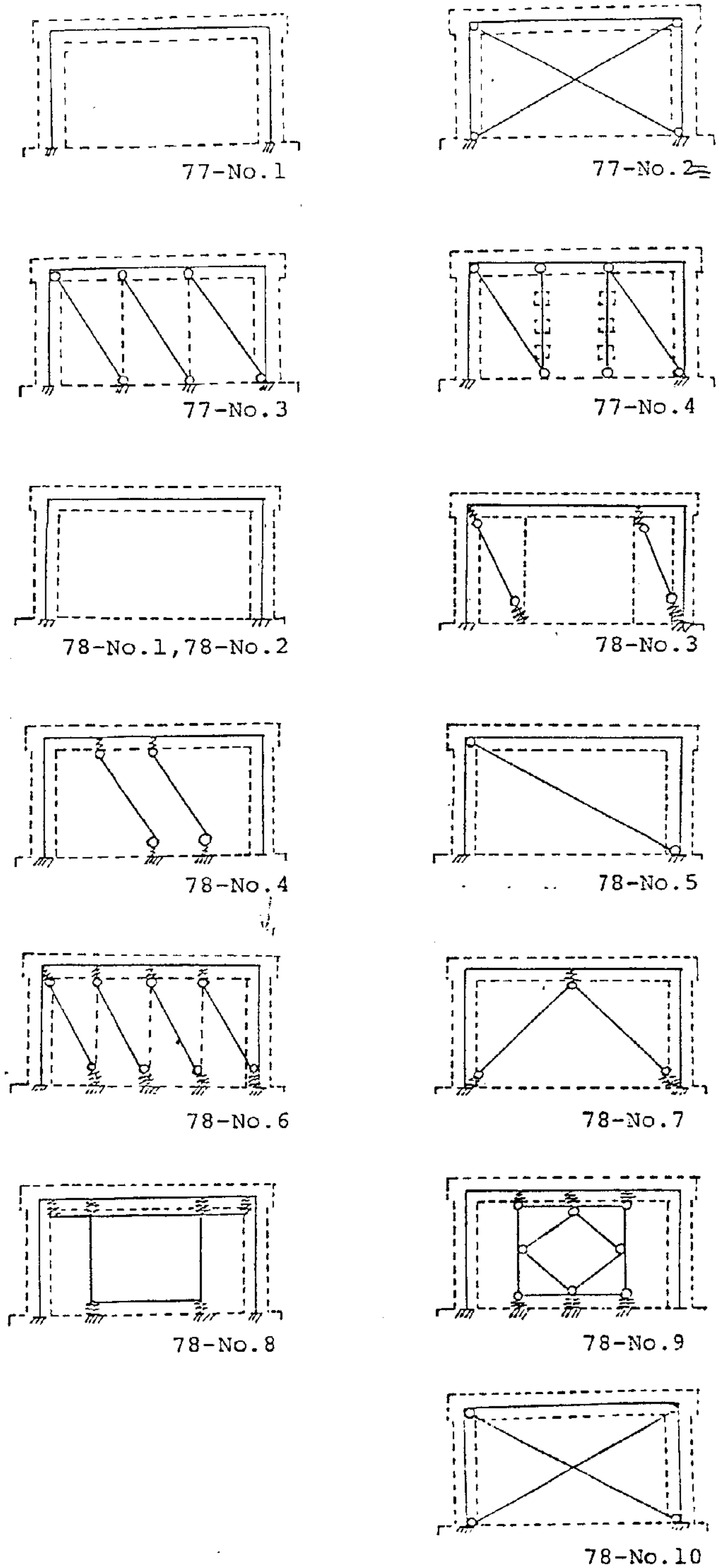
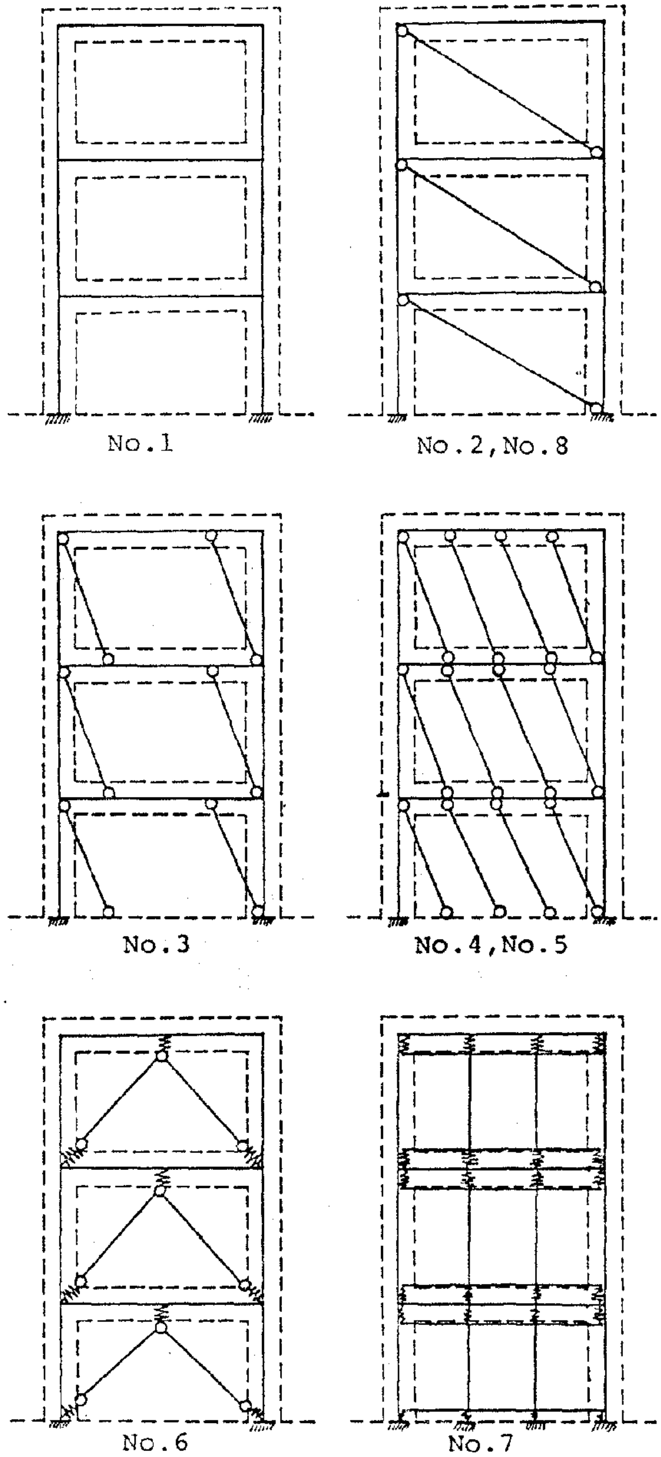
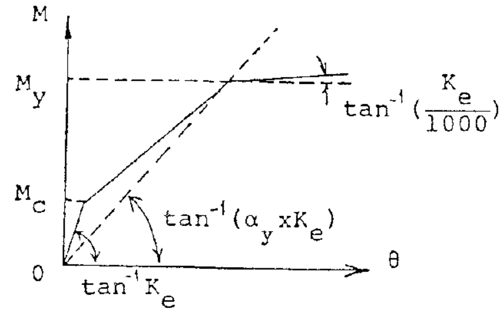


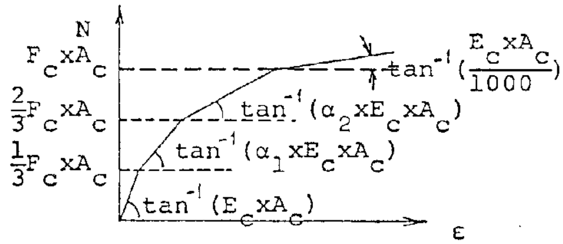
Figure 8. Displacement Model



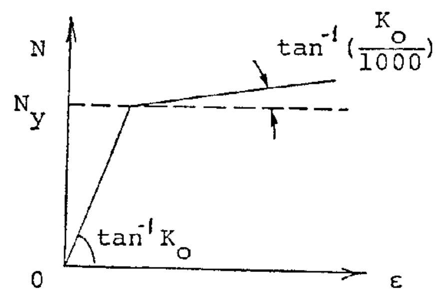
79 Series



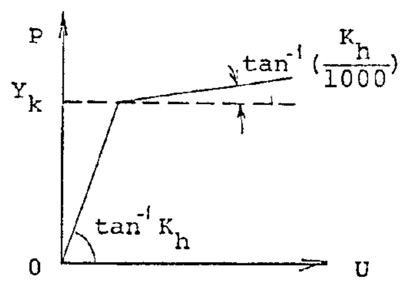
A) Moment and Rotation Relation at the Ends of Members



B) Stress-Strain Relation of the Additional Wall as Bracing



C) Strain and Axial Load Relation of the Steel



D) Load and Deflection Relation of Spring

A_c; sectional area of member

E_c; young's modulus of concrete

$N_y = \begin{cases} \text{compression;} & {}_s\sigma_c x A_s \\ \text{tension;} & {}_s\sigma_y x A_s \end{cases}$

${}_s\sigma_c$; buckling strength

A_s; sectional area of steel

Y_k; shearing strength of ancher bolt

K_e, K_o, K_h; elastic rigidity of each member

Figure 9.

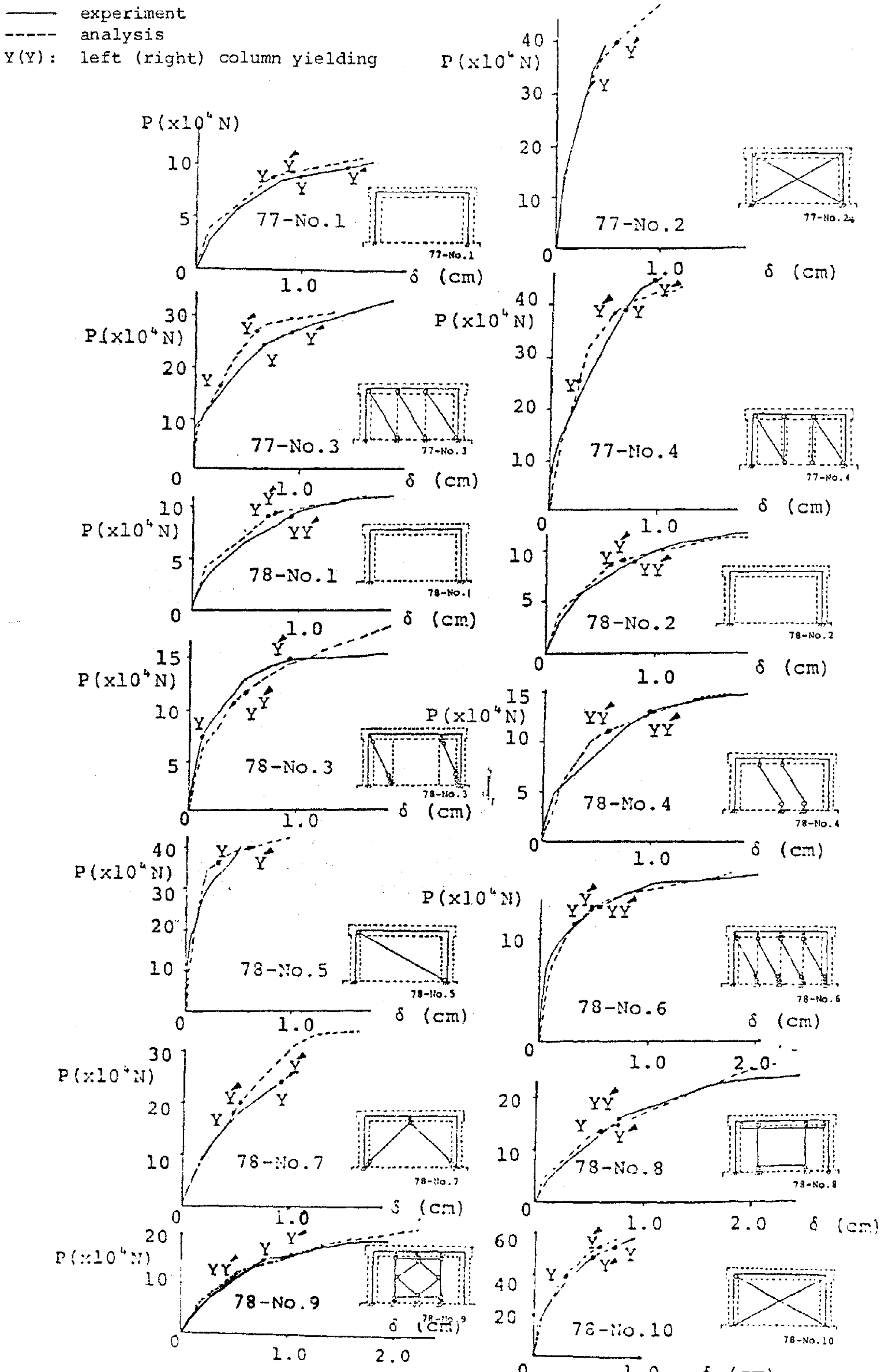
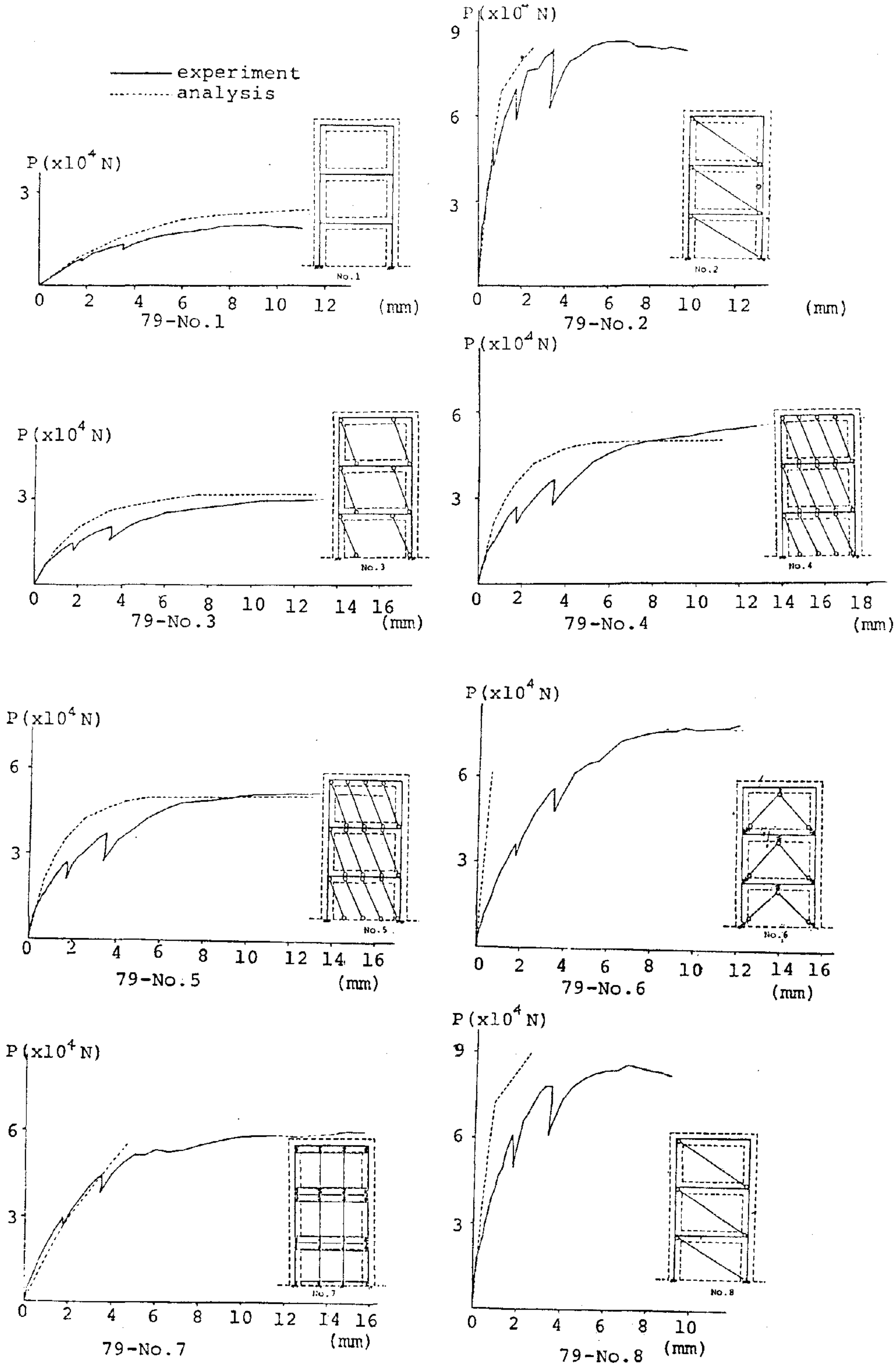


Figure 10. Result of Analysis



experiment
analysis
P(x10⁴N)
(mm)
79-No.1
79-No.2
79-No.3
79-No.4
79-No.5
79-No.6
79-No.7
79-No.8
No.1
No.2
No.3
No.4
No.5
No.6
No.7
No.8

79 series

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EPOXY REPAIR CONCRETE COMPONENTS UNDER FIRE EXPOSURE

by

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ABSTRACT

The strength properties and other important parameters regarding the behavior of epoxy repaired concrete walls during fire exposure were experimentally determined. Both ASTM E-119 and the SDHI time-temperature "pseudo" fire exposures were considered in evaluating the fire ratings of epoxy repaired concrete walls. Conclusions are provided regarding the interrelationship between fire performance and wall thickness, crack width, duration and intensity of fire exposure and the type of epoxy adhesives.

INTRODUCTION

Epoxy adhesives have been used extensively in the repair and rehabilitation of concrete, wood and masonry structures damaged by wind or earthquakes and deterioration produced by aging, environmental exposure or a variety of other causes. Excellent adhesion bonding characteristics and low shrinkage during the curing process are two primary reasons for utilizing epoxy adhesives for such repair and rehabilitation work. Since a variety of epoxy adhesives are commercially available, optimum material properties and application techniques may be chosen for different repair or rehaibilitation jobs (5, 6, 7).

The behavior under room temperature environments of epoxy repaired concrete, both under laboratory and actual field conditions, is well documented in literature (8,10,11,13). However, experimental data has not been previously available on the behavior of epoxy repaired structural components during and after typical fire exposures. Ref. 1 presented experimentally derived strength properties of pure epoxy adhesives subjected to elevated temperatures. These results indicated, as shown in Fig. 1, that above 400°F, the hot strengths are negligible for all organic epoxy adhesives currently utilized in the repair and rehabilitation of structural components. Ref. 1 also presented a theoretical finite element analysis of the expected strengths of epoxy repaired concrete components subjected to pseudo-fire exposures. This paper will present additional experimental test results on epoxy repaired concrete components in conjunction with the experimental and analytical studies described in Ref. 1.

Concrete beams, columns and walls have been repaired or rehabilitated with epoxy adhesives for nearly 30 years in the United States. Many concrete structures transfer lateral wind or seismic loads through bearing or non-bearing shear walls. The experimental results described herein pertain to the strength properties of epoxy repaired concrete shear walls during and after "pseudo-type" building fire exposures. The specimens were fabricated in three different sizes as indicated in Fig. 2 and labelled as small, intermediate and large-scale specimens. All experiments presented herein have been conducted at the Structures Laboratory of California State University, Long Beach and the University of California, Berkeley. Before discussing these experimental studies on epoxy repaired concrete shear walls, a brief summary of the strength properties of pure epoxy adhesives at elevated temperatures is presented.

STATIC STRENGTH PROPERTIES OF EPOXY ADHESIVES AT ELEVATED TEMPERATURE

A series of experimental tests were conducted on pure epoxy adhesives subjected to elevated temperatures as described in Refs. 1 and 2. An electric convection oven was used for uniform temperature control and all loads were applied statically. Relative to the experimental studies described in subsequent sections, only compression tests are described. For compressive strength tests, the test procedure including the loading rate and specimen geometry (cylinders with 1/2" diameter and 1" length) was obtained from ASTM D-695 "Test for Compressive Properties of Rigid Plastics", with the following exceptions. Each cylindrical specimen was placed in the pre-heated electric oven for a period of one hour at a specified uniform temperature. For the "hot test", the specimens were removed from the oven and immediately subjected to a static compressive load. Curve I in Fig. 1 illustrates the "hot test" results for static compressive strength. Beyond 400°F (204.4°C), the "hot" compressive strength is negligible due to cracking and rubber-like behavior of the specimens which results in reduced strength properties. The "residual test" specimens for static compressive strength were also subjected to a one-hour temperature exposure, cooled under laboratory conditions for about seven days, and subsequently tested in pure compression at room temperature. Curve II in Fig. 1 provides the "residual test" results for static compressive strength. For temperature exposures of up to 300°F (149°C) the "residual compressive strength" did not change appreciably. Beyond 400°F (204.4°C) temperature exposures, the specimens usually cracked and became rubber-like resulting in lower "residual" compressive strength properties. Since these compressive tests on pure epoxy adhesives utilized laterally unconfined specimens, the "residual" strength properties of structural epoxy adhesives confined within thin cracks may be considerably different from those indicated in Fig. 1 especially at temperatures near and above 400°F (204.4°C).

Curve I in Fig. 1 also illustrates a drastic change in the mechanical properties in the temperature range of 200°F (93.3°C) to 250°F (121.1°C). Due to the sudden drop in the "hot" strength properties at a temperature of about 230°F (110°C), this temperature is herein defined as the strength transition temperature, T_s. Curve II also shows that the maximum residual strength is achieved at temperatures near the strength transition temperature (230°F)(110°C) rather than the heat distortion temperature (136°F) (57.8°C). These results are substantiated by the thermodynamic concepts of cure or polymerization which state that the optimum post cure temperatures are above the glass transition temperature.

EPOXY CONCRETE SPECIMEN PREPARATION

Fig. 2 provides the dimensions for all small (14 in. x 18 in.), intermediate (34 in. x 40 in.) and large (90 in. x 102 in.) scale specimens. The most important specimen parameters studied include wall thickness, h, and crack width, w. The specimens were constructed with wall thickness of 6 in. (15.24 cm), 8 in. (20.32 cm), and 10 in. (25.4 cm). The crack widths studied included 0.05 in. (1.27 mm), 0.10 in. (2.54 mm), and 0.25 in. (6.35 mm).

The specimens were fabricated from ready mixed concrete using a 6 bag mix. Rounded aggregate with a 3/4 in. (19.1 mm) maximum size and Type I Portland Cement were used for the construction of all specimens. Control cylinders were prepared and tested in accordance with ASTM C39 "Standard Method of Test for Compressive Strength of Cylindrical Concrete Specimens". The average 28 day compressive strength of the control cylinders was 4.15 ksi (28.6 MN/m^2) with a standard deviation of 0.36 (2.5 MN/m^2) ksi. Fire surface protection, such as plaster, is not considered herein but is discussed in Ref. 2.

All wall specimens were cured for approximately seven days prior to the formation of the crack. To simulate actual crack surfaces of concrete shear walls, each wall specimen was broken as a beam at an angle θ equal to 45° as shown in Fig. 2. Since compression loads were applied to the top and bottom surfaces (ABFE and CDGH in Fig. 2), this crack configuration provided maximum shear stresses within the epoxy repaired crack. The concrete shear wall specimens, having been broken into halves, were cured for a minimum of at least 90 days prior to epoxy injection. The cracked specimens were cured under laboratory conditions, that is, temperature of 70°F (21.1°C) and relative humidity of 50%. After the 90 day curing period, the specimens were injected with appropriate epoxy adhesives as described in the following sections.

MOISTURE CONTENT STUDIES

Due to the significant effect of moisture on fire studies of concrete components (9, 12) moisture contents were determined for all specimens. All moisture contents were obtained by weight at a temperature of 200°F (93.3°C) for a period of approximately 10 hours (ASTM D-2016). The moisture contents were generally obtained within 3 days prior to fire exposure. The small-scale specimens were cured in a laboratory environment (70°F (21.1°C) and 50% relative humidity) for total time periods not less than 140 days but not more than 180 days prior to fire exposure. The moisture content of concrete in small-scale specimens varied from 1.4% to 1.8%. The intermediate and large-scale specimens were constructed and cured under laboratory conditions for a period of about 90 days. Subsequently, the specimens were transported to UC Berkeley Richmond Field Station by truck. These specimens were placed outside of the laboratory and covered with polyethlene film and plywood. Despite this protection, several rain storms resulted in extensive absorption of moisture by the concrete. The moisture content in concrete varied from 2% to 3% in the intermediate scale specimens. For the large-scale specimens, the moisture content in concrete varied from 2.5% to 3.3%.

EPOXY ADHESIVES USED IN THE EXPERIMENTAL PROGRAM

Six different structural epoxy adhesives were considered in this research program as described in Ref. 2. All six epoxy adhesives are considered thermosetting resins derived from the oil refining intermediate products; epichlorhydrin and bisphenol A. Fillers were not added to the epoxy adhesives either before or during the injection of the adhesives into cracks. These six epoxy adhesives were chosen because their chemical and physical properties are representative of most epoxies that have been or are being used for the repair of damaged structures since the 1971 San Fernando Earthquake. Based on technical data provided by the manufacturers of these six epoxy adhesives and additional experimental work on the physical properties of these epoxy adhesives at the Structures Laboratory, all six epoxy adhesives have been divided into two groups: low viscosity and high viscosity epoxy adhesives. The range of mechanical properties for low viscosity epoxy adhesives are as follows:

Viscosity (cps)	300 -	500
Compressive Strength at 70°F (21.1°C)(psi)(MN/m^2)	12,000 - (82.8 MN/m^2)	17,000 (117.3 MN/m^2)
Tensile Strength at 70°F (21.1°C)(psi)(MN/m^2)	7,000 - (48.3 MN/m^2)	12,000 (82.8 MN/m^2)
Pot Life (Minutes)	20 -	40
Heat Distortion Temperature (°F)(°C)	120 - (48.9)	145 (62.8)
Strength Transition Temperature (°F)(°C)	220 - (104)	240 (116)

The range of mechanical properties for high viscosity epoxy adhesives are as follows:

Viscosity (cps)	12,000 - 17,000
Compressive Strength at 70°F (21.1°C) (psi)(MN/m^2)	13,000 - 16,100 (89.7) (111.1)
Tensile Strength at 70°F (21.1°C) (psi)(MN/m^2)	6,500 - 7,800 (44.9) (53.8)
Pot Life (Minutes)	30 - 50
Heat Distortion Temperature (°F)(°C)	115 - 135 (46.1) (57.2)
Strength Transition Temperature (°F)	230 - 245 (110) (118.3)

Considerable variation in the strength and viscosity of the high and low viscosity epoxy adhesives did not affect fire test results because the heat distortion and the strength transition temperatures were similar for both types of epoxies. In this paper, the fire test results for both the low and high viscosity epoxy adhesives are averaged into a single group of test results. However, Ref. 2 provides separate fire test results for the low and high viscosity epoxy adhesives.

EPOXY INJECTION PROCEDURE AND EPOXY CURING

The epoxy resin and hardner for all epoxy adhesives were mixed together in proportions specified by the respective manufacturers. The hardner and resin were mixed together in quantities of up to 12 oz. (340.2 gm) with the aid of a high speed drill. The epoxy was either injected into the cracks at pressures below 200 psi (1.4 Mn/m^2) or simply poured into the crack whenever possible. All cracks were sealed with reinforced plastic tape and casting plaster which were both completely removed when the epoxy adhesive had cured. Since the cracked surfaces for all concrete specimens were formed as described earlier, cleaning of the cracks was not required. At the time of the epoxy injection, all cracks were dry. Prior to any type of experimental testing, all epoxy adhesives were allowed to cure for a minimum of seven days. Visual observations accompanied by hardness tests for some specimens were used to insure proper curing of the epoxy adhesives.

DESCRIPTION OF ASTM AND SDHI FIRE EXPOSURES: HOT STRENGTH AND RESIDUAL STRENGTH

The epoxy repaired shear wall specimens described in Fig. 1 were subjected to "pseudo-fire" exposures designed to simulate two different types of building fires. The two-hour duration ASTM E-119 fire exposure (4) for shear walls attempts to model a long duration fire with constantly increasing temperature, so that the cool down behavior is not represented. A short duration high intensity (SDHI) fire which peaks at about 0.2 hours, has a rapid temperature drop for a period of 0.4 hours and is followed by a slow cooling to room temperature. This SDHI time-temperature curve has been proposed by Professor Boris Bresler of U.C. Berkeley and others (Ref. 3). Both the ASTM and the SDHI time-temperature curves are provided in Fig. 3. As indicated by the results in subsequent chapters, the ASTM E-119 type fire exposure is far more severe than the SDHI type on the fire rating of epoxy repaired structures. Temperatures were recorded on the unexposed face EFGH with results provided in Ref. 2 and summarized in the following sections.

During fire exposure, the small-scale specimens were not subjected to any type of external loads. However, upon completion of the fire exposure, "hot strength" and "residual strength" compression tests were conducted. "Hot strength" type of tests refer to epoxy repaired concrete shear wall specimens which were subjected to ultimate compression loads about 10 minutes after the end of fire exposure. "Residual strength" tests refer to epoxy repaired specimens which were subjected to prescribed fire exposure, allowed to cool in a laboratory environment (70°F (21°C) and 50% relative humidity) for a period of seven days, and then subjected to ultimate compression loads. As indicated later, "residual strengths" of epoxy repaired shear walls were signficantly higher as compared to "hot strengths". The strength properties of pure epoxy adhesives at elevated temperatures as given in Fig. 1, provide the explanation for the lower "hot strength" as compared to "residual strength" test results. The "residual strength" tests are designed to evaluate the strength properties of epoxy repaired shear walls after the building fire and the "hot strength" tests during the building fire.

EXPERIMENTAL TEST RESULTS FOR SMALL SCALE SPECIMENS

The following sections provide a brief explanation of the individual test procedures and test parameters followed by test results. The subsections are divided according to type of fire exposure and the nature of the compression test, that is "hot" or "residual". In a future article, the effects of fire surface coatings such as plaster, on the behavior of epoxy repaired shear walls during fire exposure will be presented.

Description of Test Procedure

All small-scale fire tests were conducted in the natural gas furnaces at California State University, Long Beach. After the specimens were fully prepared, that is, the injected epoxy had been cured for a minimum of seven days, the specimens were placed in the furnace with only surface ABCD in Fig. 2 exposed to the fire.

During the fire exposure, bearing loads were not applied to the small-scale specimens. Immediately after the fire exposure, the specimens were removed from the furnace and subjected to the ultimate compression load until failure in the case of "hot strength" tests. The depth of epoxy burnout or pyrolysis within the crack was determined for each specimen immediately after the specimen had failed under compression loading. The "residual strength" tests were conducted according to the test procedure described earlier for compression tests on pure epoxy cylinders. All of the following stress data is given in terms of the applied load divided by the gross cross-sectional area of plane ABEF in Fig. 2. For the small-scale specimens, the average depth of concrete spalling adjacent to the epoxy repaired crack varied from about 0.6 in. (1.3 cm) to 1 in. (2.54 cm) for both the two-hour ASTM E-119 and the one-hour SDHI fire exposures.

Small-Scale ASTM E-119 Hot Strength Compression Tests

This section provides a summary of test results for small-scale specimens whose dimensions and load application are described in Fig. 2. All test specimens were exposed to the standard two-hour ASTM E-119 fire exposure for walls. Primary test parameters studied in this section include crack widths of 0.05 in. (1.3 mm), 0.10 in. (2.5 mm) and 0.25 in. (6.3 mm). All specimens were subjected to ultimate compression loads about 10 minutes after the end of the two-hour fire exposure. This loading procedure yields the lowest strength values for epoxy repaired shear walls as indicated by the test results for intermediate and large scale specimens in later sections. Figs. 4 and 5 provide the test results for average ultimate compressive strength and depth of epoxy burnout as a function of wall thickness and crack width. The failure pattern for all specimens, including 6 in. (15.2 cm), 8 in. (20.3 cm) and 10 in. (25.4 cm) shear wall specimens, consisted of shear failure in the epoxy since the temperatures within the specimens during the compression tests were above the heat distortion temperatures. Ultimate compressive stress is a function of crack width due to the development of higher frictional forces resulting from aggregate interlock in the case of smaller crack widths. Depth of epoxy burnout is approxiately 3 inches and not significantly affected by crack width. Note that the ultimate compressive stress is not significantly increased by increasing wall thickness from 6 in. to 10 in. as indicated in Fig. 4.

Small-Scale SDHI Hot Strength Compression Tests

This section provides a summary of test results for small-scale specimens whose dimensions and load application are described in Fig. 2. All test specimens were exposed to the standard one-hour SDHI fire exposure with time-temperature curve given in Fig. 3. Primary test parameters studied include crack widths of 0.05 in. (1.3 mm), 0.10 in. (2.5 mm), and 0.25 in. (6.4 mm) and wall thicknesses of 6 in. (15.2 cm), 8 in. (20.3 cm), and

10 in. (25.4 cm). All specimens were subjected to ultimate compression loads about 10 minutes after the end of the fire exposure.

Figs. 6 and 7 provide the test results for average ultimate compressive stress and depth of epoxy burnout as a function of wall thickness and crack width. Ultimate compressive stress as in the previous section, is a function of crack width due to the development of higher frictional forces resulting from aggregate interlock in the case of smaller crack widths. Depth of epoxy burnout is about 1 inch and is not significantly affected by crack width. The failure pattern for the 6 in. (15.2 cm) thick wall specimens consisted of shear failure in the epoxy since the temperatures within these specimens during the compression tests were generally above the heat distortion temperature. The failure pattern for most 8 in. (20.3 cm) and 10 in. (25.4 cm) shear wall specimens generally consisted of combined shear failure within concrete and epoxy in regions where the epoxy was not burned out. Note that the depth of epoxy burnout for the SDHI fire is much less than for the ASTM fire. However, the compressive strengths are not significantly different when comparing Figs. 4 and 6.

Small-Scale ASTM E-119 Residual Strength Compression Tests

This section provides a summary of test results for small-scale specimens whose dimensions and load application are described in Fig. 2. All test specimens were exposed to the standard two-hour ASTM E-119 fire exposure for walls. Primary test parameters studied in this section included crack widths of 0.05 in. (1.3 mm), 0.10 in. (2.5 mm), and 0.25 in. (6.4 mm) and wall thicknesses of 6 in. (15.2 cm), 8 in. (20.3 cm), and

10 in. (25.4 cm). All specimens have been subjected to ultimate compression loads seven days after the two-hour fire exposure. Loads were not applied during fire exposure.

Figs. 8 and 9 provide test results for average ultimate compressive stress and depth of epoxy burnout as a function of wall thickness and crack width. Compressive strength is extrapolated for 10 in. thick walls. Depth of epoxy burnout is about 3 in. and is not significantly affected by crack width. The failure pattern for the 6 in. (15.2 cm) thick wall specimens consisted of a combined shear failure in epoxy and concrete. For the 8 in. (20.3 cm) thick wall specimens, the failure pattern consisted primarily of a shear failure in concrete.

Fig. 10 provides a pictorial view of a small-scale 6 in. (15.2 cm) thick shear wall specimen exposed to a two-hour ASTM E-119 fire and subjected to a "residual" compressive strength test. The right side is the fire exposed face. The light region, approximately 1 in. (2.54 cm) wide adjacent to the fire face, represents primarily concrete spalling during the fire exposure. The black region approximately 2 in. (5.1 cm) wide, represents the charred epoxy region where complete pyrolysis of the epoxy had occurred. The light region adjacent to the unexposed or the back face represents shear failure in the concrete. The dark transverse line near the middle of this last region represent the epoxy which had not been pyrolyzed during the fire exposure. Note that the failure was primarily through the concrete not the epoxy in this last region.

Small-Scale SDHI Residual Strength Compression Tests

This section provides a summary of test results for small-scale specimens whose dimension and load application are described in Fig. 2. All of these test specimens were exposed to the standard one-hour duration SDHI fire exposure for walls. Primary test parameters studied include crack widths of 0.05 in. (1.3 mm), 0.10 in. (2.5 mm) and 0.25 in. (6.4 mm) and wall thicknesses of 6 in. (15.2 cm), 8 in. (20.3 cm) and 10 in. (25.4 cm). All specimens have been subjected to ultimate compression loads seven days after the fire exposure. Loads were not applied during the fire exposure.

Figs. 11 and 12 provide the graphical summary of average test results including average ultimate compressive stress and depth of epoxy burnout as a function of crack width and wall thickness. The failure pattern for all specimens consisted of shear failure in the concrete. Compressive strength is extrapolated for 10 in. thick walls. Depth of epoxy burnout is not significantly affected by crack width and is approximately 1 in. as in the corresponding "hot strength" tests in Fig. 7.

INTERMEDIATE AND LARGE-SCALE SPECIMENS AND THEIR TEST RESULTS

Eight intermediate size and two large-scale specimens were fabricated and experimentally tested under fire exposure as in the case of the small-scale specimens described in the previous sections. The properties of epoxy and concrete were identical to those for small-scale specimens. Fig. 2 illustrates the specimen geometry and Table 1 provides a partial test matrix and test results. The primary purpose of this expensive intermediate and large-scale testing program was to investigate the effect of specimen size on fire test results. Table 1 does not include four specimens that were fabricated with fire protective coatings including plaster and both organic and inorganic thin fire protection coatings.

All specimens in this section were tested for "hot" compressive strength properties at the University of California, Richmond Field Station, Fire Test Laboratory. The fire tests were conducted according to ASTM E-119 test procedure or the SDHI time-temperature curve given in Fig. 3. Except for the presence of nominal bearing loads and specimen size differences of height and width, all other test parameters and test procedures were identical to those described in the previous sections for small-scale specimens. In Table 1, large-scale specimens are labelled G-9 and G-10 and the remaining specimens are of intermediate size.

The tabular matrix shown in Table 1 provides the specimen number and specimen thickness in the first two columns respectively. Crack widths of 0.10 in. and 0.25 in were tested since the effects of fire on the 0.05 in. wide epoxy repaired cracks were less severe as indicated in the small-scale test results. Both the two-hour ASTM E-119 and the one-hour SDHI time-temperature curves were utilized to model "pseudo" building fires as noted in column 4 of Table 1. The applied compressive stress during fire exposure on side ABEF (see Fig. 2) is given in column 5. These stress values were chosen primarily on the basis of the loading capacity of the testing frame. Time of failure is provided in column 6 relative to the starting of the fire exposure. No failure in column 6 indicates that under the applied compressive stress given in column 5, the wall specimen did not fail structurally during or after fire exposure. The maximum unexposed face temperature recorded during fire exposure was measured according to ASTM E-119 test procedure and is shown in column 7. Average depth of concrete spalling adjacent to the epoxy repaired cracks is given in column 8. The extent and depth of such spalling is affected significantly by the moisture content in the concrete as verified by a comparison of moisture contents for the small, intermediate and large-scale specimens.

Fig. 13 provides a cross-sectional view of the epoxy repaired crack after the fire exposure. The extent of concrete spalling and the depth of epoxy burnout are illustrated by the indicated nomenclature provided with this figure. The depths of spalling and epoxy burnout are uniform along the cracks except for minor edge effects at the ends of the cracks. Fig. 13a pertains to the large-scale specimen G-9 which was subjected to a two-hour ASTM E-119 fire exposure. Fig. 13b shows a cross-sectional view of the epoxy repaired crack after a one-hour SDHI fire exposure for intermediate size specimen, G-4.

COMPARISON OF TEST RESULTS FOR SMALL, INTERMEDIATE AND LARGE-SCALE SPECIMENS

Comparison of Strengths

All small-scale tests were conducted with ultimate loads applied about 10 minutes after the end of fire exposure ("hot strength tests") or more than seven days after the end of fire exposure ("residual strength tests"). The intermediate and large-scale specimens were subjected to a compressive stress of 220 psi and 110 psi respectively during and after the end of fire exposure with times of failure provided in Table 1. If failure did not occur during fire exposure, the lowest strengths were obtained between 5 to 15 minutes after the end of fire exposure as indicated by specimen G-2. Based on the times of failure and the ultimate strength values provided in Table 1 and in previous graphs, the small, intermediate and large-scale specimens yield very similar strength results.

Comparison of Epoxy Burnout

Table 1 provides average total depth of epoxy burnout for intermediate and large-scale specimens. Comparison with small-scale specimens, the burnout depth for the two-hour ASTM E-119 fire exposure is generally about 3 inches and about 1 inch for the one-hour SDHI fire exposure. Note that the epoxy burnout is not significantly affected by wall thickness or crack width.

Comparison of Thermal Gradients

E-119 type fire exposures for walls theoretically generate thermal gradients only through the wall thickness. Hence, the temperature on the unexposed face of the walls are independent of the specimen length and width dimensions. As discussed in a previous section, the moisture content in concrete can significantly affect the temperatures within the wall and on the unexposed face. The moisture contents for the small-scale specimens were generally lower than for the intermediate and large-scale specimens. Hence, the maximum temperatures on the unexposed face of the small-scale specimens were from 5% to 10% higher than for the intermediate and large-scale specimens given in Table 1.

CONCLUSIONS

The following conclusions are based on test results given in previous sections for small, intermediate, and large-scale epoxy repaired wall specimens illustrated in Fig. 1. These conclusions are generalized, for more specific details refer to appropriate graphs and tables in Ref. 2. Definitions for "hot" and "residual" strengths, along with other terms, are provided in earlier sections.

1. Good to excellent comparison was obtained for test results of small, intermediate and large-scale specimens. Therefore, the cheaper small-scale specimens can be effectively used to study the behavior and mechanical properties of epoxy repaired concrete walls subjected to fire exposure rather than the expensive full-scale specimens described in ASTM E-119 specifications.

2. The mechanical properties of all epoxy adhesives currently used for the repair of concrete structures are very similar at temperatures about 100°F above the heat distortion temperatures (110°F (43.3°C) to 150°F (65.6°C)). Since the thermal gradients in walls up to 10 in. (25.4 cm) thick generate temperatures above 200°F (93.3°C) throughout most of the wall thickness, the behavior and strength properties of various epoxy adhesives in epoxy repaired concrete walls during and after fire exposure are very similar.
3. Crack width, wall thickness, type of fire exposure, type of stresses applied on epoxy repaired crack, and the presence or lack of presence of fire protection coatings are the primary parameters affecting the strength properties and behavior of epoxy repaired cracks in concrete walls during and after fire exposure.
4. The duration and intensity of fire exposure has great significance on the strength and behavior of epoxy repaired concrete walls both during and after fire exposure. In this research program, the standard two-hour ASTM E-119 and the one-hour SDHI fire exposures (see Fig. 3) were used to investigate the significance of duration and intensity. For unplastered specimens, the compressive strength properties for the SDHI fire were about two times greater than for the ASTM fire (compare Figs. 8 and 10). Similarly, the depth of epoxy burnout was about three times greater for ASTM fires in comparison to SDHI fires.
5. For a two-hour ASTM E-119 or the one-hour SDHI fire, "hot strength" properties of epoxy repaired concrete walls from 6 in. (15.2 cm) to 10 in. (25.4 cm) thickness (except for thin epoxy injected cracks subjected to pure compressive stresses) are reduced to levels far

below the original design stress levels. For example, for an 8 in. (20.3 cm) thick concrete wall with epoxy injected diagonal cracks as in Fig. 2 from 0.05 in. (1.27 mm) to 0.25 in. (6.35 mm) wide and subjected to a two-hour ASTM E-119 fire, the compressive "hot strength" will vary from 200 (1.38 MN/m^2) to 600 psi (4.14 MN/m^2).

6. The direction of the epoxy repaired crack in relation to the applied stresses has significant effect on the strength properties of epoxy repaired components during fire exposure. For thin epoxy repaired cracks subjected to normal compressive stresses, strength reduction is minimal. Epoxy repaired cracks subjected to parallel shear stresses may suffer total loss of strength depending on the thermal gradients, crack width and extent of aggregate interlock.

7. In conjunction with Conclusions 5 and 6, the loads which an epoxy repaired concrete component must transfer during a fire need to be carefully considered. For shear walls, the simultaneous occurrance of a fire and a severe earthquake or wind load is not realistic. Thus, the investigation for the strength properties and behavior of epoxy repaired concrete walls during a fire should consider only the presence of dead loads and live loads other than severe lateral wind or seismic loads.

8. Most "residual strength" properties of epoxy adhesives subjected to elevated temperatures (but not burned or pyrolyzed) are increased more than 50% (see Fig. 1). Therefore, the unburned epoxy adhesive remaining in the crack after a fire exposure will possess higher strengths than prior to a fire exposure.

9. The lowest strength properties of epoxy repaired concrete walls do not occur during fire exposure but rather five to fifteen minutes after the end of both the E-119 and SDHI fire exposures (See test results for intermediate-scale specimens in Table 1). This phenomenon is due to (1) the presence of thermal gradients causing increasing temperatures at and near the unexposed face after the end of fire exposures, and (2) rapidly decreasing strengths of epoxy adhesives at temperatures above 230°F (110°C) with near zero strengths at temperatures above 400°F (204°C).

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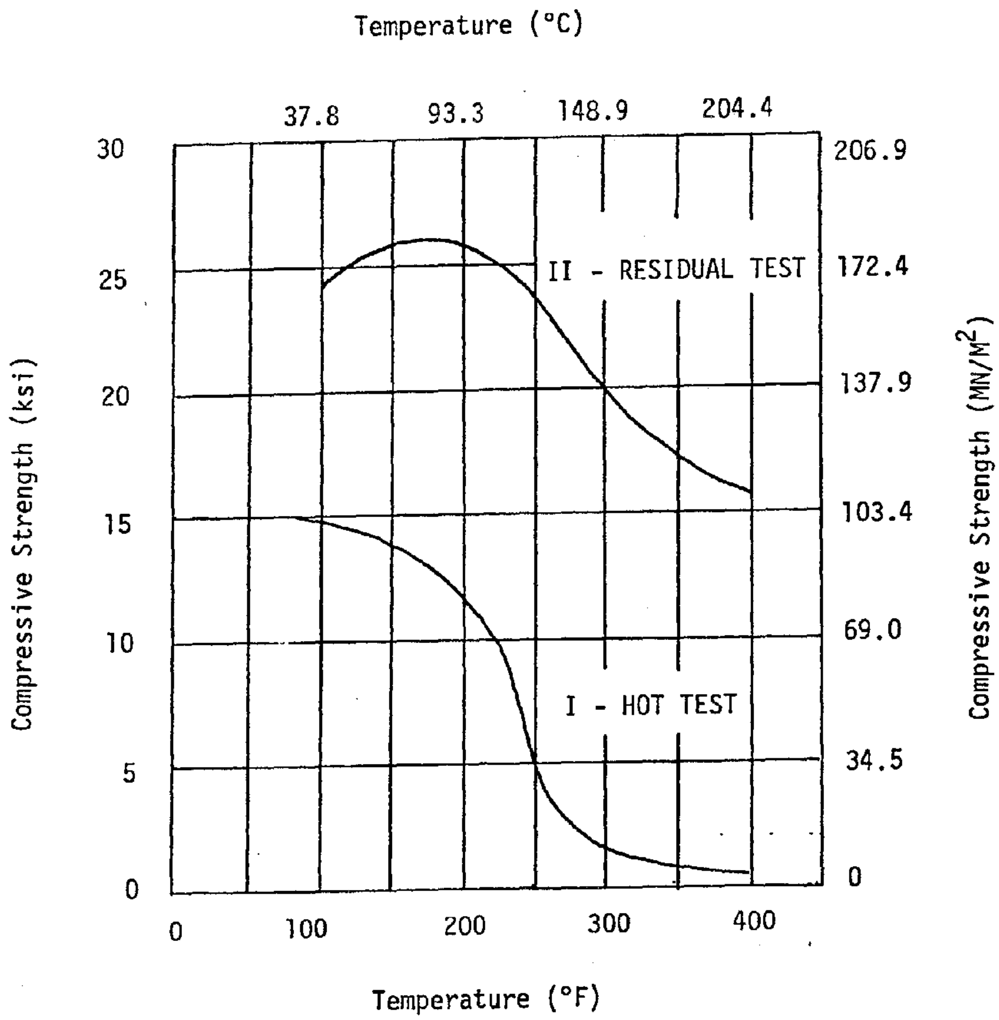


Figure 1: Compressive Strength of Epoxy Adhesives

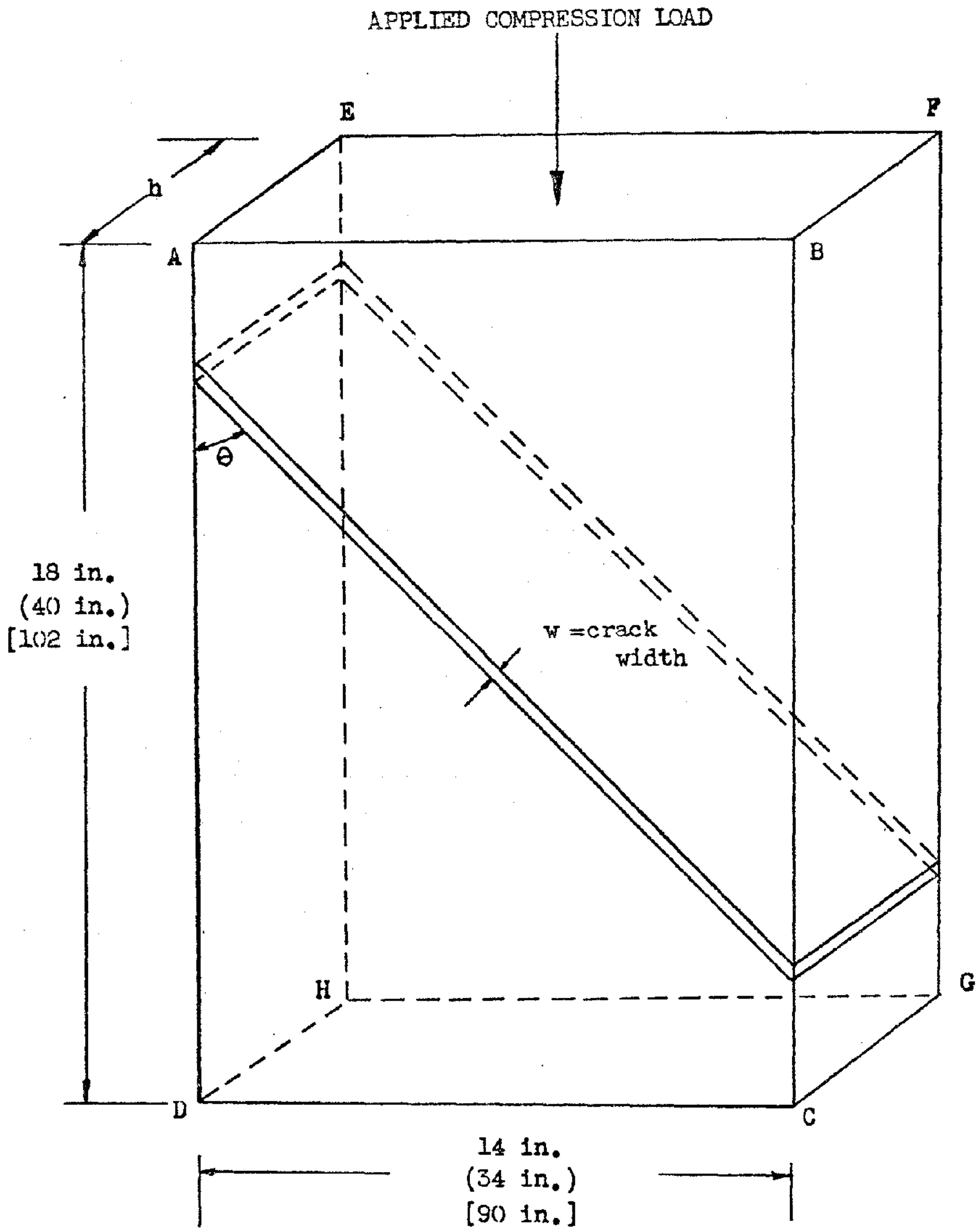


Fig. 2: General Specimen Configurations

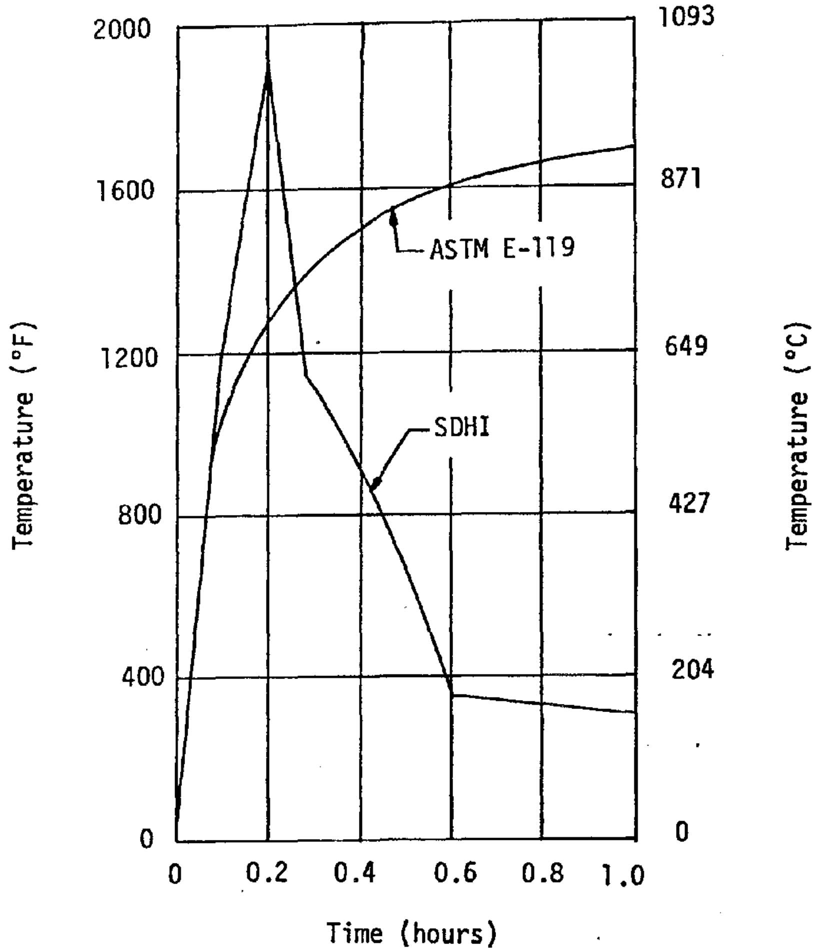


Fig. 3: ASTM E119 and SDHI Time-Temperature Fire Curves

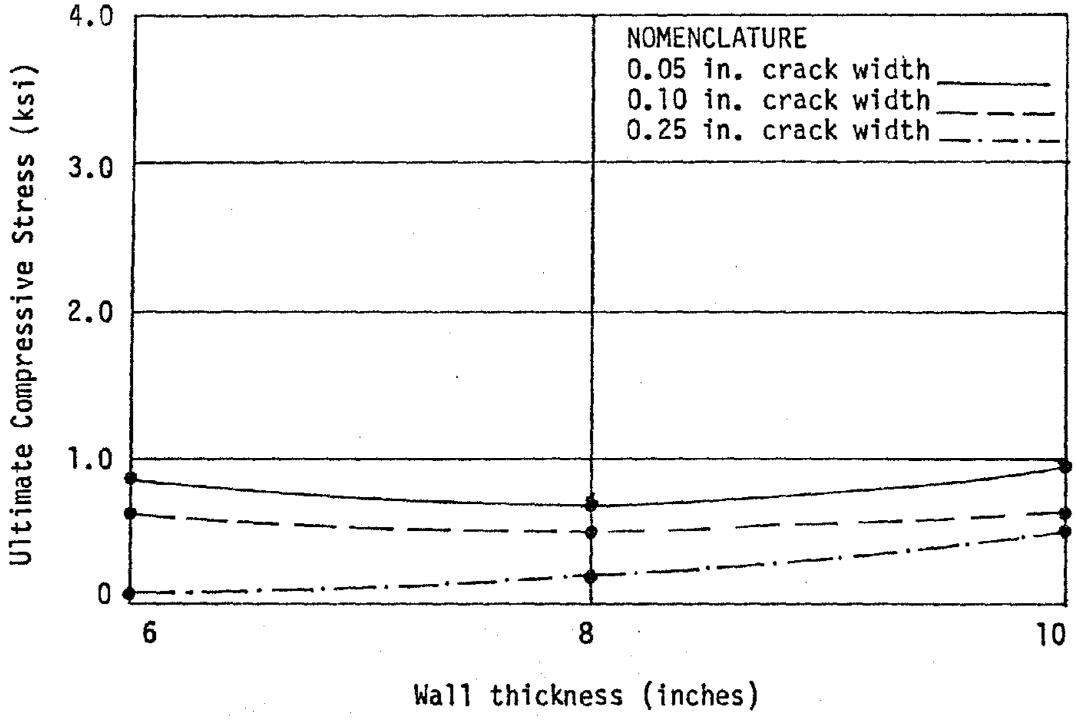


Fig. 4 : Average "Hot" Compressive strength as a Function of Wall Thickness for Small-Scale ASTM E-119 Fire Exposure

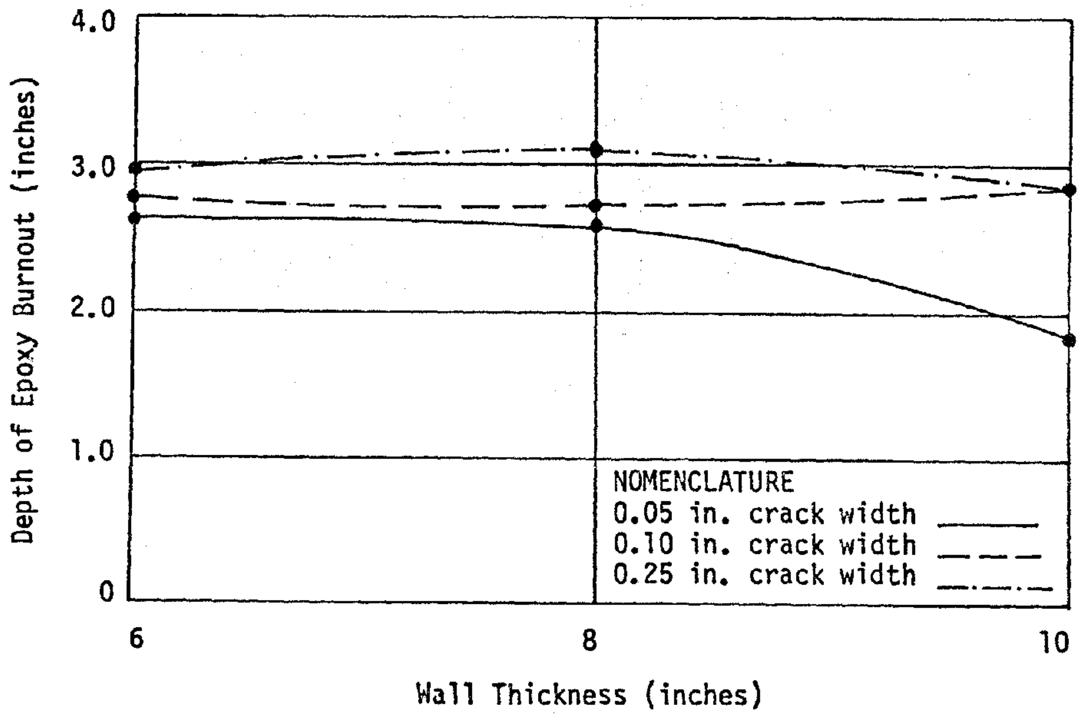


Fig. 5 : Average Depth of Epoxy Burnout as a Function of Wall Thickness for Small-Scale ASTM E-119 Fire Exposure.

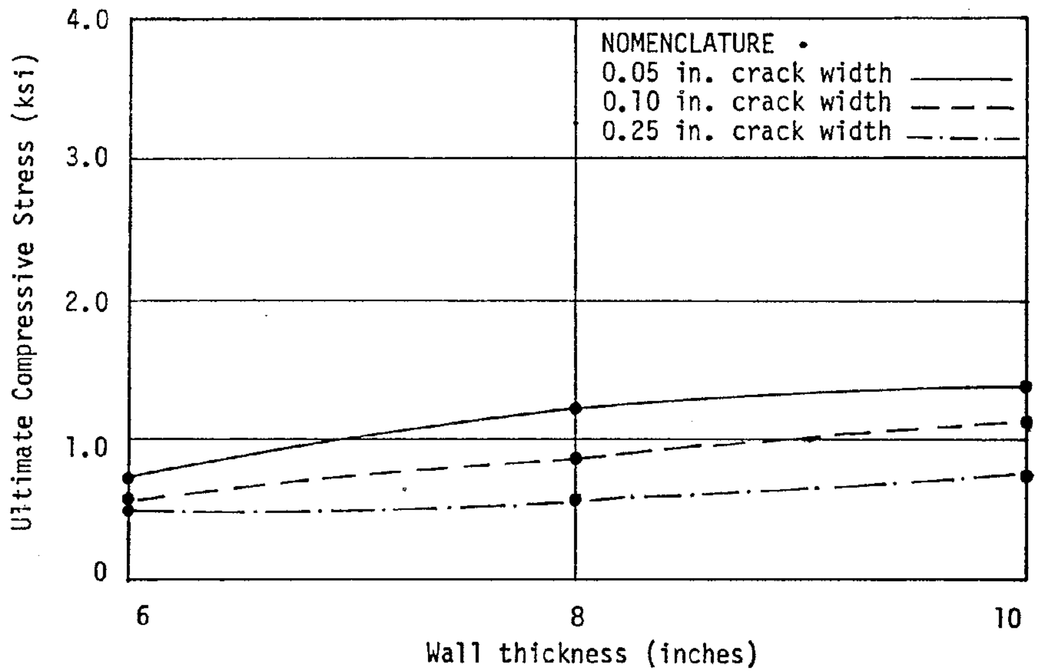


Fig. 6 : Average "Hot" Compressive Strength as a Function of Wall Thickness for Small-Scale SDHI Fire Exposure

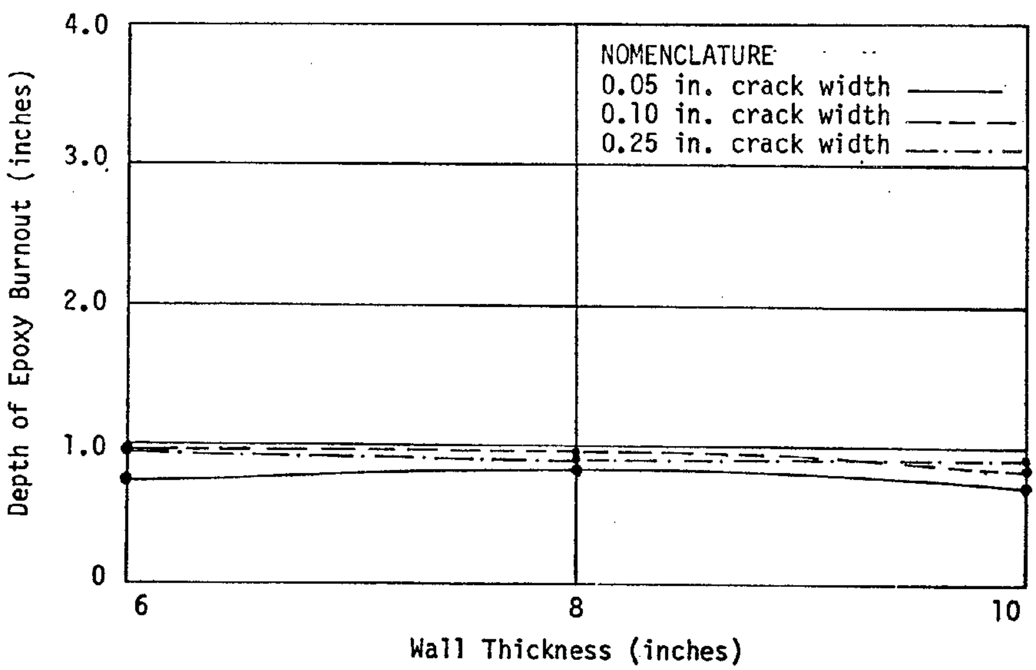


Fig. 7 : Average Depth of Epoxy Burnout as a Function of Wall Thickness for Small-Scale SDHI Fire Exposure

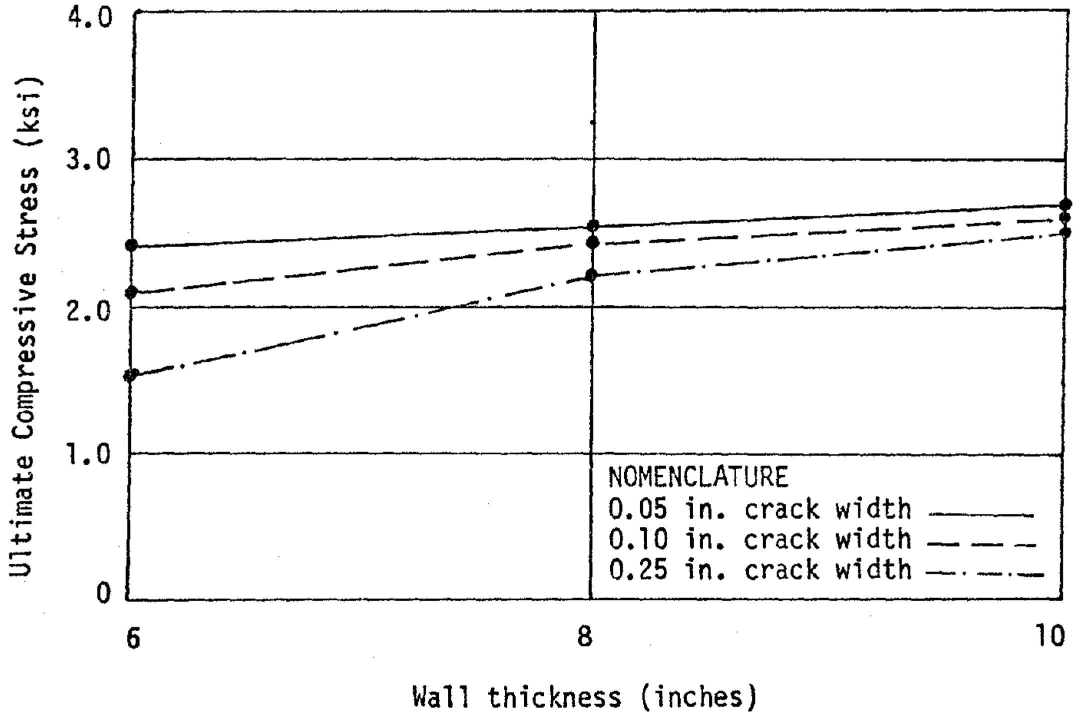


Fig. 8 : Average "Residual" Compressive Strength as a Function of Wall Thickness for Small-Scale ASTM E-119 Fire Exposure

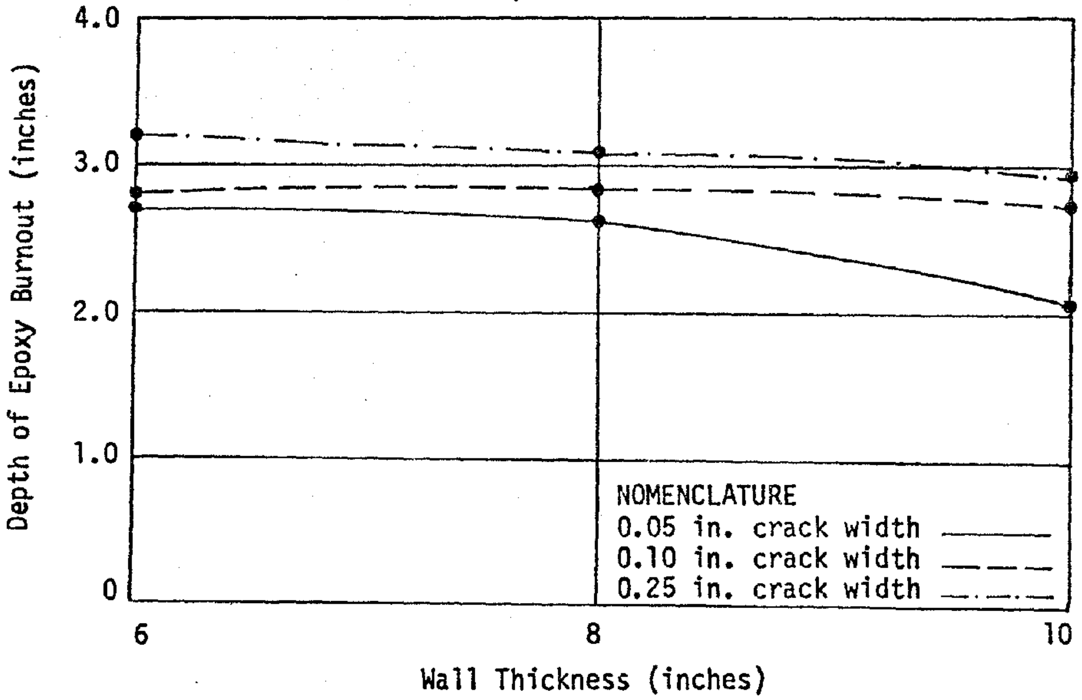


Fig. 9 : Average Depth of Epoxy Burnout as a Function of Wall Thickness for Small-Scale ASTM E-119 Fire Exposure

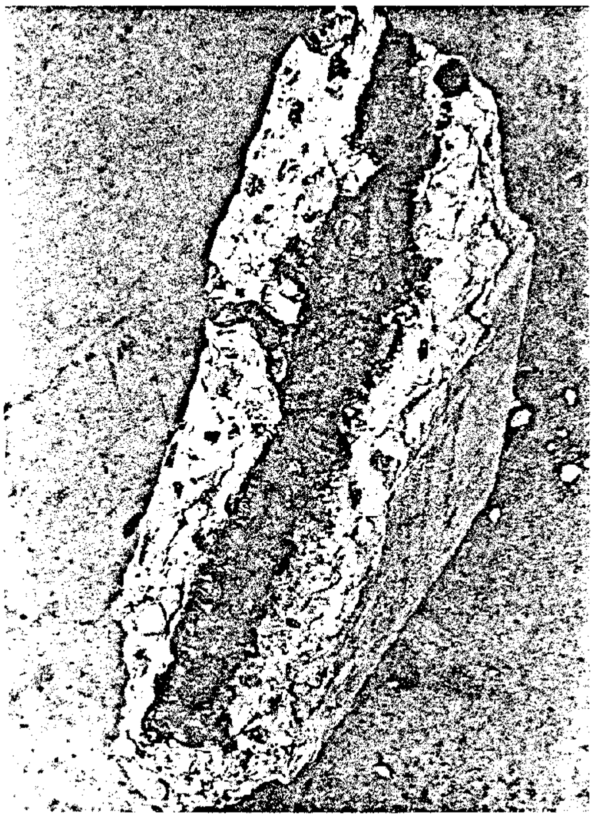


Fig. 10: View of 6 in. Thick Shear Wall Specimen after a "Residual" Strength Compression Test

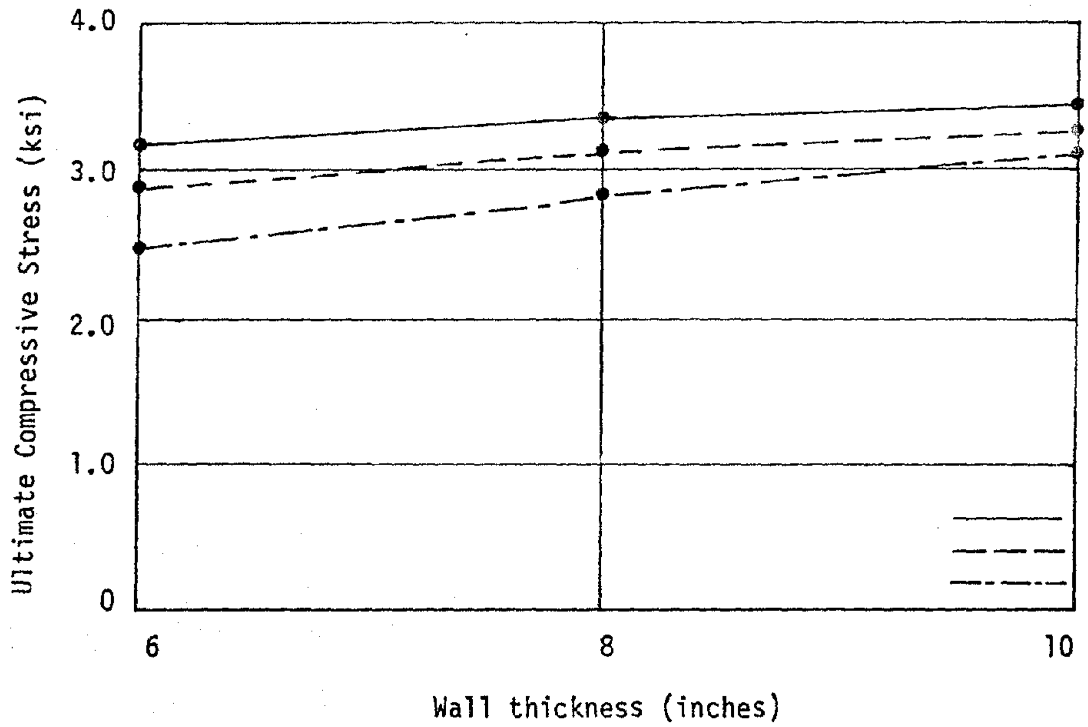


Fig. 11 : Average "Residual" Compressive Strength as a Function of Wall Thickness for Small-Scale SDHI Fire Exposure

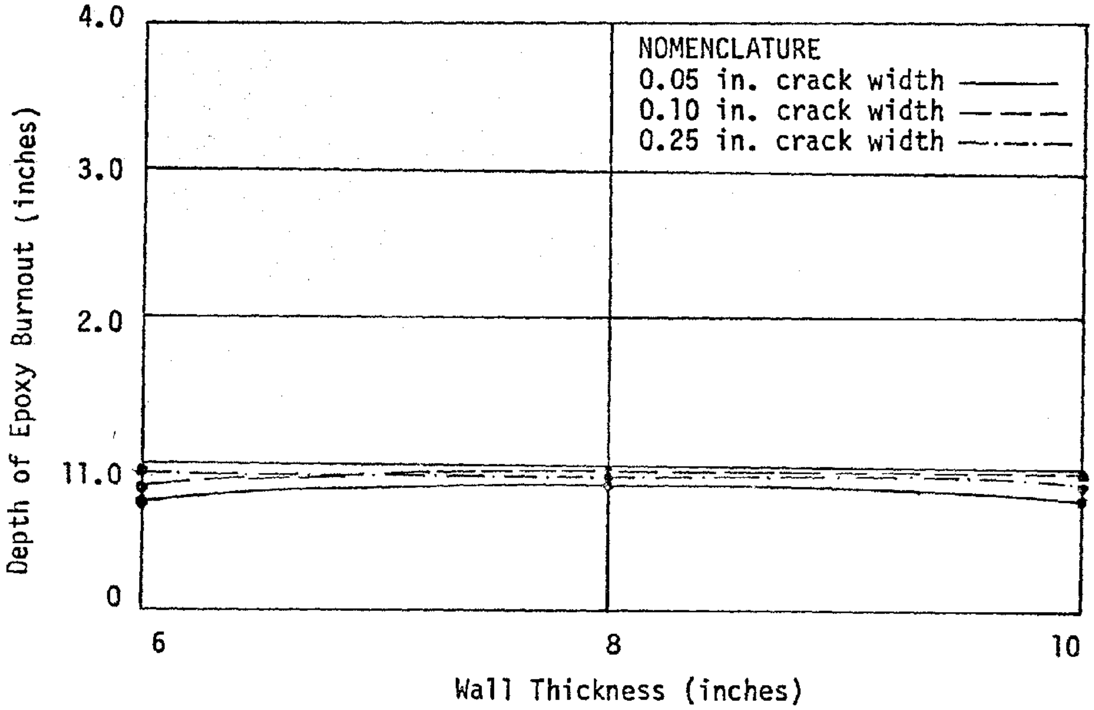
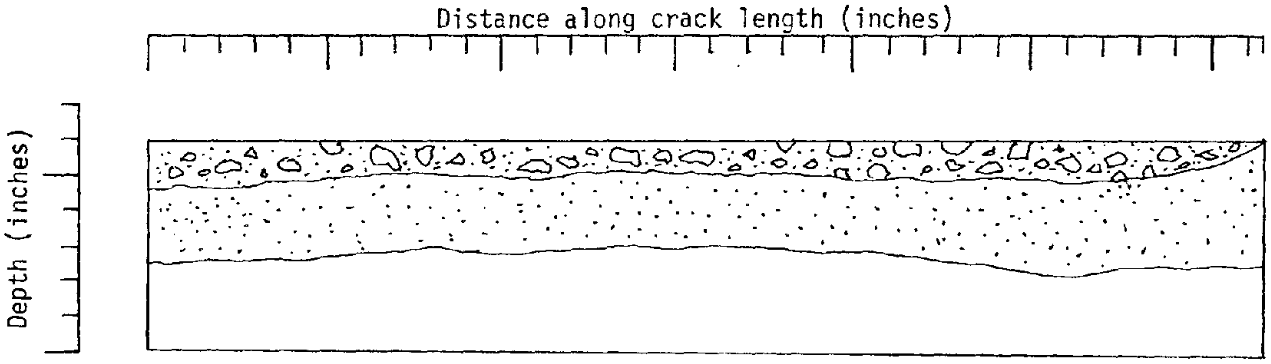
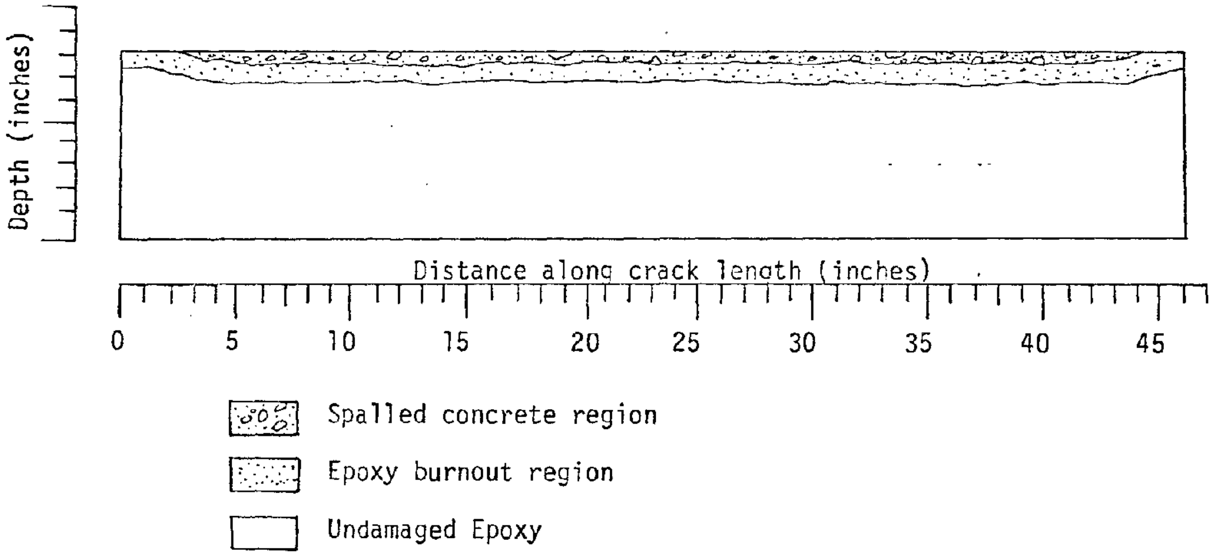


Fig. 12 : Average Depth of Epoxy Burnout as a Function of Wall Thickness for Small-Scale SDHI Fire Exposure



(a) View along half of diagonal crack for large-scale specimen G-9



(b) View along diagonal crack for Intermediate scale specimen G-4

Fig.13. Cross-sectional view along Epoxy Repaired Concrete Wall specimens after Fire Exposure

TABLE 1: TEST RESULTS FOR INTERMEDIATE AND LARGE-SCALE SPECIMENS

Specimen Number	Nominal Concrete Thickness (Inches)	Crack Width (Inches)	Time Temperature Curve	Applied Comprehensive Stress (psi)	Time of Failure (Minutes)	Maximum Temperature: Unexposed face (°C)	Average Depth of Spalling (Inches)	Average Depth of Epoxy Burn-out (Inches)
G-2	10	0.25	ASTM	220	133.25	35	1.0	2.6
G-3	8	0.10	SDHI	220	No Failure	17	0.5	.9
G-4	8	0.10	SDHI	220	No Failure	20	0.6	1.0
G-8	8	0.10	ASTM	220	No Failure	Not Available	0.75	3.1
G-9*	6	0.25	ASTM	110	50	88	1.0	3.0
G-10*	8	0.25	ASTM	110	80	61	1.3	3.1

* G-9 and G-10 are large-scale specimens, the others are of intermediate size.

GUIDELINE FOR SEISMIC RETROFITTING (STRENGTHENING, TOUGHENING AND/OR STIFFENING) DESIGN OF EXISTING REINFORCED CONCRETE BUILDINGS

by
S. Sugano

PREFACE

This note describes the outline of the "Guideline for Seismic Retrofitting (Strengthening, Toughening and/or Stiffening) Design of Existing Reinforced Concrete Buildings" which was proposed by the advisory committee (Chairman, Professor H. Umemura, University of Tokyo) for the Ministry of Construction, Japanese Government, on March, 1977. The guideline was published in Japanese by the Japan Building Disaster Prevention Association.

The guideline was translated into English by K. Yagishita (Toda Kensetsu Co., Ltd.), and the material was compiled and arranged by S. Sugano as his supplemental lecture note for the "Seminar on Seismology and Earthquake Engineering" at Building Research Institute, Tsukuba, on March 13 - April 12, 1980. The commentary for the guideline is not included herein, however, some of the important figures shown in the guideline or in its commentary are attached at the end of each chapter for reference. Foot notes are also provided in each chapter for the convenience of reffering the figures.

CONTENTS

1. GENERAL

1.1 Scope

This guideline is applied to the seismic retrofitting (strengthening, toughening and/or stiffening) design and construction of existing reinforced concrete buildings and their non-structural elements.*1 In the case of design based on a special investigation or research, this may not be applied. The matters not described herein, however, shall follow the "Standard for Design of Reinforced Concrete Structures" of the Architectural Institute of Japan (AIJ) and the "Japanese Architectural Standard Specification (JASS)" of AIJ.

1.2 Aim of Seismic Retrofitting Design

In the seismic retrofitting design, the aim of seismic capacity shall be distinctly established.*2, *3

1.3 Preliminary Investigation

The seismic retrofitting design shall be based on sufficient investigation of the objective building.

1.4 Plan of Retrofitting

In the seismic retrofitting design, the basic plan should be shaped and the adequate construction method should be selected on the basis of synthetic discussions on results obtained from the evaluation of the seismic safety and from the preliminary investigation, on the change of function within the building by the retrofitting, and on the feasibility of the construction.*4

1.5 Evaluation of Retrofitting

The evaluation of seismic capacity of retrofitted structural elements is in principle performed in accordance with the methods shown in each section of Chapter 2. The Evaluation of the effect of retrofitting on the overall behavior of the building is in principle based on the "Criterion on the Evaluation of Seismic Safety of Existing Reinforced Concrete Buildings". The influence of retrofitting upon unstrengthened structural parts should be also examined.

For the cases with retrofitting methods not described herein, those shown in Section 2.4, and those with particularly detailed connections, the calculation and evaluation of retrofitting should be accomplished in principle on the basis of adequate experimental informations following the philosophy of this guideline.

1.6 Construction for Retrofitting

The construction of seismic retrofitting is performed in accordance with the methods shown in each section of Chapter 3.

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*1 The flow chart of design and construction of seismic retrofitting is shown in Fig.1.1.

*2 The following values are recommended as the aim of seismic capacity.

i) ${}_{R}I_{S} \geqq 1.2\ I_{SO}$

ii) ${}_{R}C \geqq 0.3$

where, ${}_{R}I_{S}$: "Seismic Index of Structure" after retrofitting

I_{SO} : Minimum value of "Seismic Index of Structure" for the judgement that the building does not need retrofitting.

${}_{R}C$: Coefficient of lateral force capacity after retrofitting (the sum of "Strength Index C" of structural members in each direction)

*3 The concept of the aim of seismic retrofitting and the type of earthquake resistance of buildings are shown in Fig.1.2.

*4 Retrofitting techniques are illustrated in Fig.1.3.

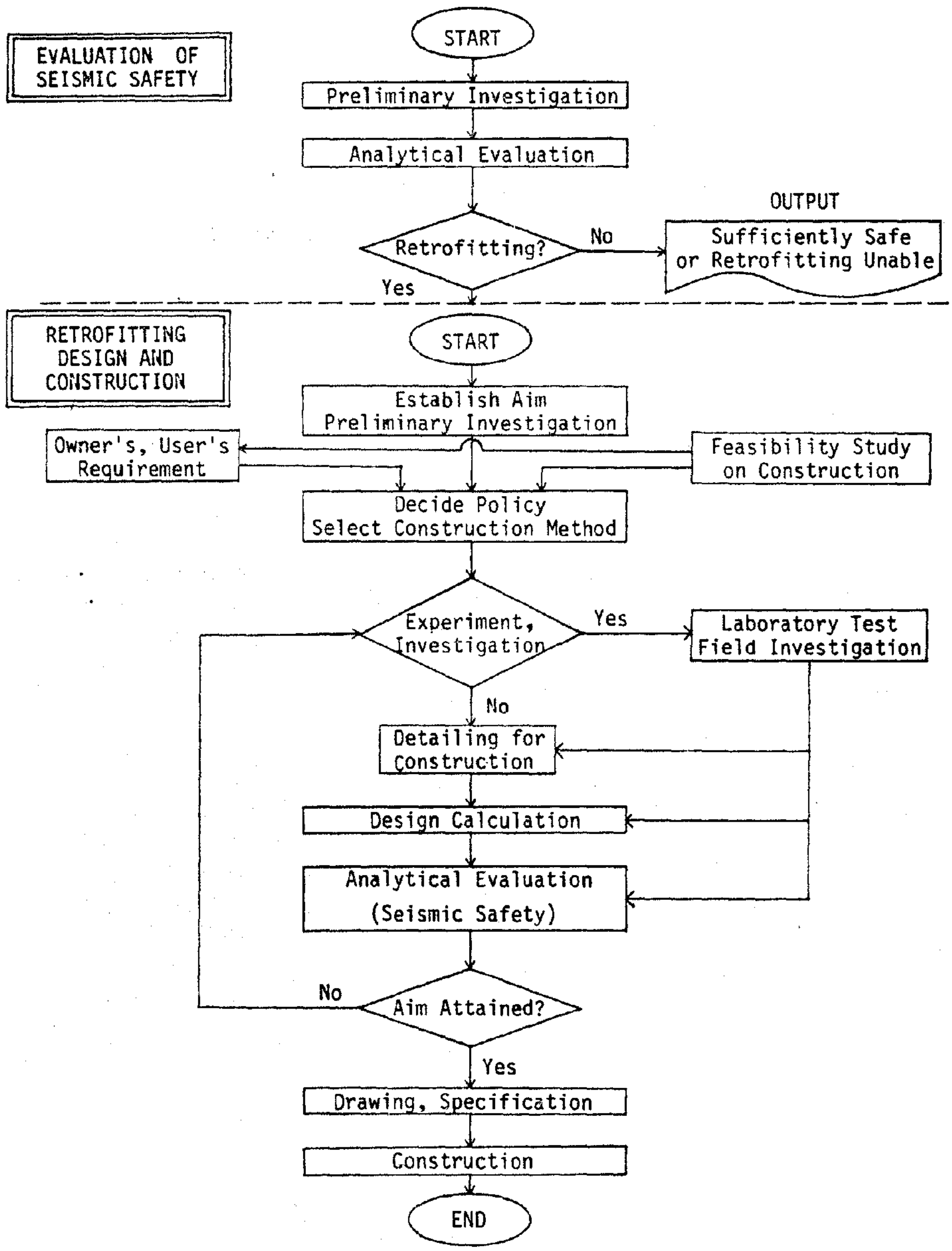
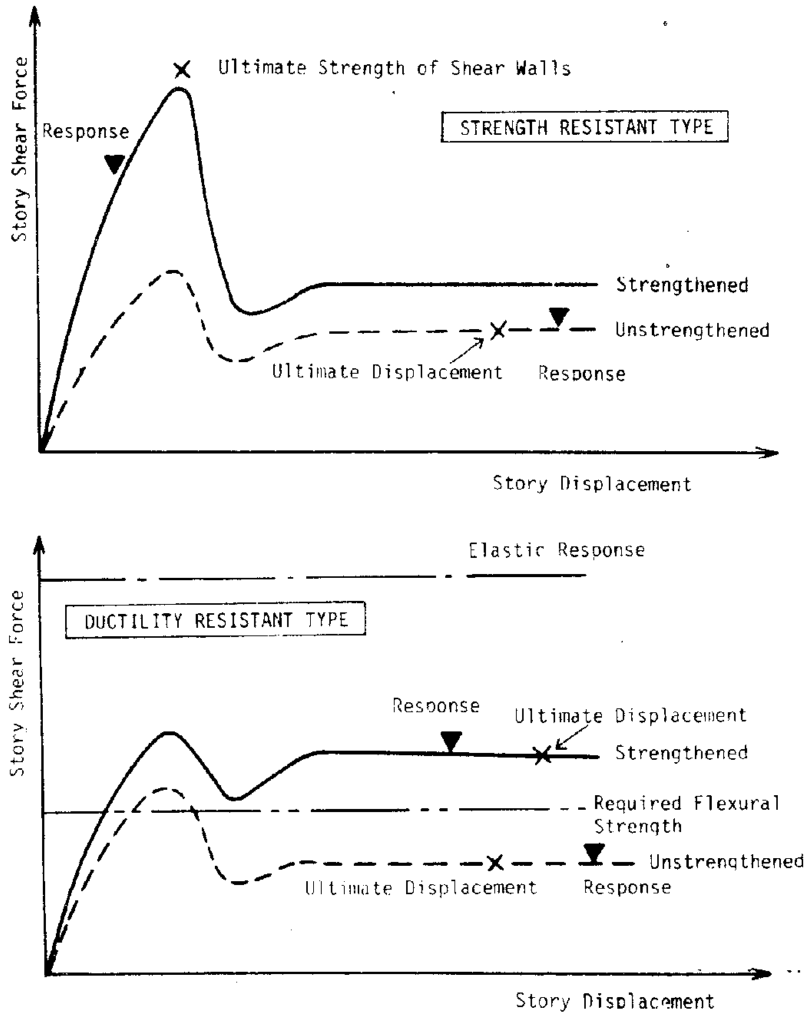
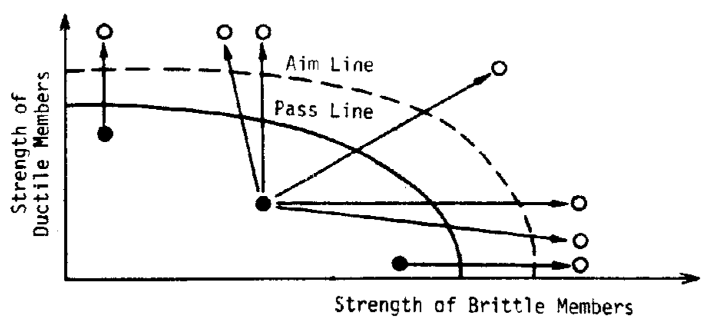


Fig. 1.1 FLOW CHART OF DESIGN AND CONSTRUCTION OF ASEISMIC STRENGTHENING

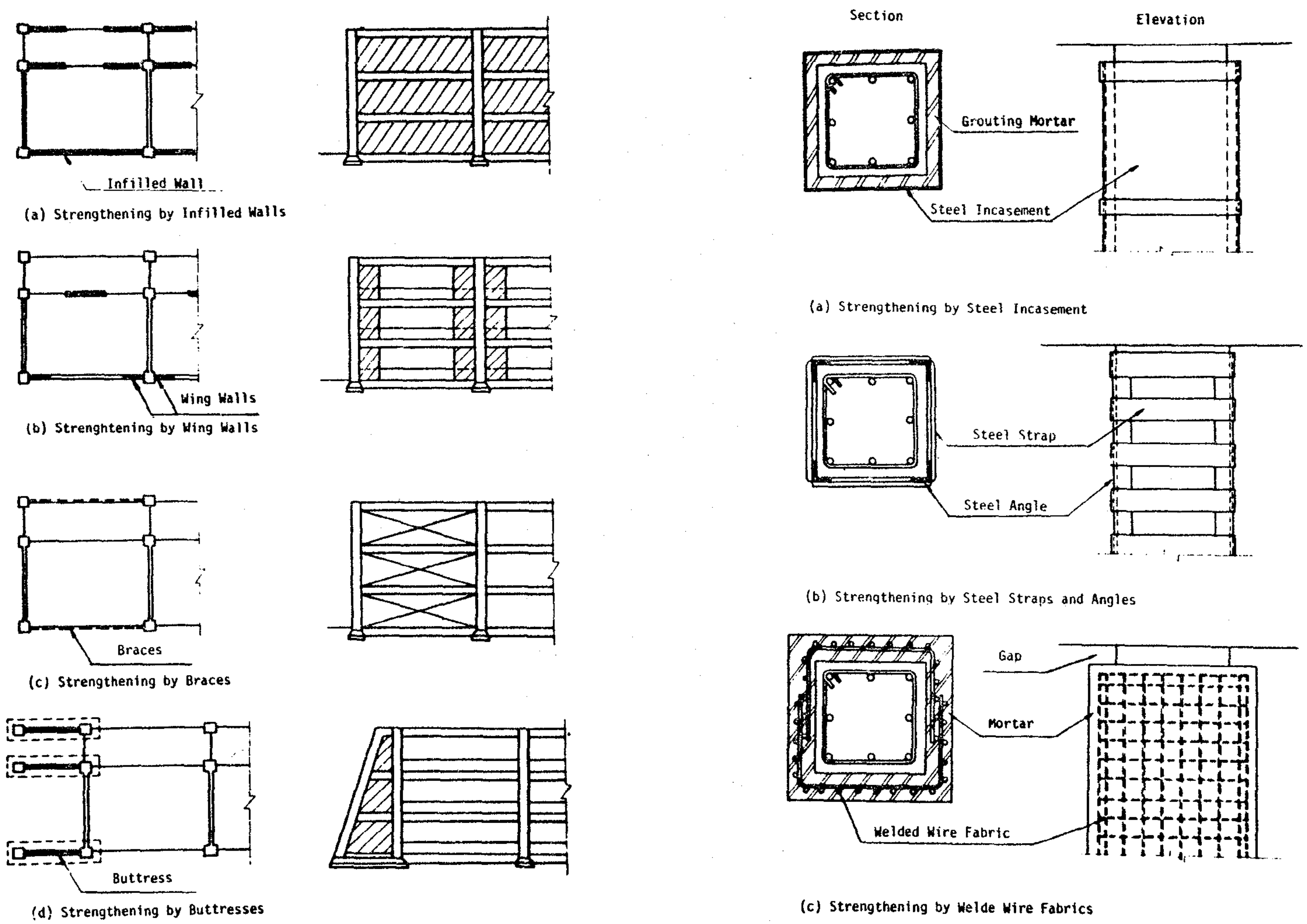


(a) Type of Earthquake Resistance



(b) Aim of Strengthening

Fig. 1.2 AIM OF ASEISMIC STRENGTHENING



(1) Techni es to Increase Strength

(2) Techniques to Increase Ductility

Fig. 1.3 CONSTRUCTION TECHNIQUES FOR STRENGTHENING

2. GUIDELINE FOR SEISMIC RETROFITTING DESIGN

2.1 Strengthening by Infilled Shear Walls

2.1.1 General

Infilling existing open frames with walls and thickening existing walls are appropriate methods to improve the lateral force capacity of a building which lacks adequate earthquake resistance. The stress between an additional wall and an existing frame should be sufficiently conveyed by using connecting element such as dowel reinforcement or shear cotter, or by developing wall reinforcement into the framing members or welding wall reinforcement with the existing reinforcement.

When these infilling techniques are selected for the design, it should be taken into account that the shear strength of an additional wall is not fully developed when the whole flexural strength of the system including surrounding frames or the overturning strength of the wall is less than the ultimate shear strength of the wall. Foundations and the supporting ground should be safe enough against the increased vertical load caused by additional walls and the change of vertical forces during an earthquake associated with the change of failure mechanism of a whole structural system due to the strengthening.

2.1.2 Aim of Seismic Capacity

(1) Capacity of Infilled Walls

Infilled shear walls should have sufficient strength so that they may increase the lateral force capacity of an unstrengthened building up to the required capacity by the design or more. However, when the adequately increased strength is not expected because of the failure mode in which the capacity of the total structure is controled of by the flexural capacity of a whole wall-frame system or overturning strength of walls, the walls should have adequate ductility capacity.

The ultimate strength of an additional wall is expressed in terms of the average shear stress along the clear length of the wall panel. The design average shear stress of a wall without opening is less than or equal to 30 kg/cm^2, and that of a wall with opening may be reduced according to the location or the area of openings. The ductility index F of a wall is classified as follows in accordance with the failure mode.

1) shear failure type 1.0

2) whole flexural failure type .... $\leqq 2.0$

3) overturning failure type 3.0

(2) Capacity of the Total Building

The aim of strengthening by infilled shear walls is to improve the lateral force capacity of the building so that it may resist acting lateral forces as "strength resistant type building" by its sufficient force capacity. However, as previously described in paragraph (1), the increased lateral force capacity of the building may not necessarily be so sufficient as expected. In such a case, the aim of strengthening is to provide adequate amount of flexural strength as well as sufficient energy absorption ability by post-yielding displacement as "ductility resistant type building".*1

2.1.3 Plan of Strengthening

(1) Objective Buildings

The buildings to which the infilling technique is effectively applied are those of which the lateral force capacity is low or those of which the seismic capacity is controled by shear failure type members. However, when the ultimate strength of additional walls is determined by the flexural or overturning strength of the wall-frame system, the technique is effectively applied to buildings of which the seismic behavior is controled by the flexural resistant type members, however, the capacity is low, utilizing their ductility capacity.

The usage and function of the building may be disturbed by placing new walls since the interior space is subdivided and natural lighting is disturbed. Therefore, the building should be allowable above matters to apply the infilling technique. Furthermore, additional walls may result in considerable increase of dead load, and may result in the change of axial forces during lateral loads associated with the change of resistant mechanism. For that reason, the building is desired to have sufficient strength of the foundation.

(2) Configuration of Additional Walls

Taking into consideration of the limitation of usage of the building, additional walls should be arranged under well-balance in both plan and elevation from the structural view point.

2.1.4 Construction Type and Structural Details*2

In accordance with the connecting methods between an additional wall and an existing frame, the type of construction is classified as follows. The structural detail common to each type is shown in the last paragraph (5).

(1) Dowel Connection Type*3

Wedge anchors are placed in the predrilled holes of the existing frame, and the shear stress between the wall and the existing frame is conveyed by the dowel action of the anchors.

Structural details:

1) Wedge anchors shall be placed within the width of concrete core of existing framing members.

2) Sufficient reinforcement against splitting shall be arranged around the dowel reinforcement in the additional wall.

3) The space of wedge anchors shall be as follows:*4

pitch : $\geq$ 7.5Dd and $\leq$ 30 cm

gage : $\geq$ 5.0Dd

edge distance: $\geq$ 2.5Dd

where, Dd is the outside diameter of bolts at the connected face.

4) Wedge anchors may be placed only on the upper and lower faces of beams.

5) The embedded length of a wedge anchor shall be in principle more than or equal to 5Dd and not less than the covering thickness.

(2) Chipped Cotter Connection Type *5

In this construction method, cotters which are formed by chipping the existing concrete transmit the shear stress between the wall and the existing frame.

Structural details:

1) Standard value of the ratio of the length to depth of a cotter is 5 : 1. The length shall be not less than 15 cm, and the width shall be not more than the wall thickness.

2) Anchor reinforcement with the diameter more than 10 mm (D10) shall be arranged within cotters in two layers.

3) The space of cotters shall be calculated based on the ratio of the strength of concrete of additional wall to that of existing frame and on the length of cotters. Cotters shall be arranged in equal spaces.

4) The number of cotters on each connecting face is desired to be more than or equal to five.

(3) Adhesive Cotter Connection Type *6

Precast concrete or mortar cotters are attached on the existing frame with epoxy resin adhesive material. The shear stress between the wall and the existing frame is transmitted by the cotters. Because of insufficient test data on the durability of adhesive materials, attention must be paid to this point in the design.

Structural details:

1) Standard value of the ratio of the length to thickness of a precast cotter is 5 : 1.

2) Adhesive parts shall be protected by the covering concrete with the thickness more than 30 mm.

3) Precast cotters shall be reinforced by using steel bars with the diameter more than 6 mmø.

4) Anchor reinforcement thicker than D10 shall be arranged in two layers in new concrete part placed between precast cotters.

5) In the horizontal connected face, dowel reinforcement having the steel ratio more than 0.25 per cent to the area of connected face shall be arranged in addition to cotters.

6) The number of precast cotters on each connected face is desired to be more than five.

(4) Other Connection Types

The following types are available besides the above mentioned connection types.

1) Welded dowel with existing reinforcement.

2) Welded dowel with mechanically anchored plate.

3) Hooked dowel on existing reinforcement.

In applying these types, careful construction is necessary in order to obtain the reliable structural capacity.

(5) Common Structural Details

The structural details common to each connection type are shown as follows.

1) The thickness of an additional wall shall be more than 1/4 the width of a column, more than 15 cm, and less than the width of a beam.

2) The shear reinforcement ratio of an additional wall shall be not less than 0.25% and not more than 1.2%. When the thickness of the wall

is more than 18 cm, the shear reinforcement shall be arranged in two layers.

3) Additional reinforcement of 2-D13 shall be arranged along the periphery of the shear wall in addition to the needed shear reinforcement.

4) The specified compressive strength of concrete of the walls shall be not less than that of existing frame.

5) When openings are provided in the wall, the peripheral reinforcement along the opening shall be designed corresponding to the required strength of the wall.

The method to provide walls may be either of the following two;

i) cast-in site method (conventional casting method or press-in method)

ii) to place precast concrete wall within the existing frame.

2.1.5 Design Calculations

(1) Process of Calculation

The process of design calculation for infilled shear walls is as follows.

1) To examine the structural capacity of the objective building.

2) To determine the design policy, in another words, to determine the type of earthquake resistance of the building, that is, strength resistant type, ductility resistant type, or their combination.

3) To establish the aim of strengthening in accordance with the design policy.

4) To assume the design stress of walls and the specified strength of materials.

5) To calculate the required wall length assuming the thickness of walls, and to determine the configuration of walls.

6) To calculate the required amount of shear reinforcement of each wall, and to design connection elements.

7) To calculate the ultimate strength of each wall.

8) To judge whether or not the aim of strengthening is satisfied.

When the aim of strengthening is satisfied and the strengthening is feasible, this calculation process is completed. However, when the aim is not satisfied or the designed strengthening is not feasible, the calculation is repeated returning to 5) or 6).

(2) Calculation of Ultimate Strength of Members

1) The ultimate strength of an infilled shear wall is the minimum value of the following i), ii) and iii).

i) Ultimate shear strength; It takes the smaller value of the following i. or ii.

i. 80% of the ultimate shear strength calculated as that of a monolithic wall cast with surrounding columns and beams.

ii. The integrated strength of the following individual strength considering the deflection mode, that is, ultimate shear strength of connection elements, punching shear strength at the end of a column, and the ultimate flexural or shear strength of a column.*7

ii) Ultimate whole flexural strength of a wall-frame system including the surrounding frame.

iii) Ultimate overturning strength of a wall-frame system including the surrounding frame.

2) Each type of ultimate strength of a member and system specified above is obtained as follows.

i) Ultimate shear strength of a monolithic wall; Eq.(13) shown in Section 3.2.2, (2), g) of "Criterion on The Evaluation of Seismic Safety" is applied.

ii) Ultimate strength of an infilled shear wall; Considering the flow of forces in connection elements, wall and columns, it is calculated by the following equation.

$$_{w}Q_{su} = \text{minimum}\ [(_{w}Q'_{su} + 2\cdot Q_c\cdot\alpha),\quad (Q_j + {}_{p}Q_c + Q_c\cdot\alpha)] \qquad (2.1.1)$$

where; $_{w}Q_{su}$: ultimate shear strength of the framed wall (t)

$_{w}Q'_{su}$: ultimate shear strength of the wall panel (t)

Q_j : sum of the ultimate shear strength of connection elements along the length of the beam (t)

$_{p}Q_c$: punching shear strength at the top of a column (t)

Q_c : the lower value of the ultimate flexural or shear strength of the other column

α : reduction coefficient relating to the deflection of a column

$$\begin{cases} 1.0 \text{ —— for shear failure of a column} \\ 0.7 \text{ —— for flexural failure of a column} \end{cases}$$

When there is a opening in the wall the strength obtained by i) or ii) is reduced based on "Standard for Design of Reinforced Concrete Structure" of AIJ. When the area of opening is larger than the specified value in the AIJ Standard, the strength is calculated as that for a column with wing walls.

iii) Ultimate flexural strength of the wall; Eq.(12) shown in Section 3.3.2, (2), c) of "Criterion on The Evaluation of Seismic Safety" is used. However, when a wall and beams are connected by wedge anchors, the strength of wall reinforcement should be less than or equal to that determined by the ultimate pull-out strength of anchors. In addition, no strength of wall reinforcement is evaluated for the case with cotter connection.

iv) Ultimate overturning strength of the wall; It is calculated in accordance with the method shown in Section 3.2.2, (2) of "Criterion on The Evaluation of Seismic Safety".

v) Ultimate frexural and shear strengths of a column, a column with wing walls, and a beam; They are calculated in accordance with the method shown in Section 3.2.2, (2) of "Criterion on The Evaluation of Seismic Safety".

vi) Ultimate punching shear strength of a column; It is calculated by the following equation.

$$ {}_{p}Q_{c} = \frac{1}{1.5} \cdot {}_{c}f_{t} \cdot b \cdot D \sqrt{1 + \frac{\sigma_0}{{}_{c}f_{t}}} \qquad (2.1.2) $$

where; ${}_{c}f_{t} \;=\; 1.8\sqrt{E_{c1}}$

F_{c1} : compressive strength of concrete (kg/cm^2)

b : width of the column (cm)

D : depth of the column (cm)

σ_0 : axial stress (kg/cm^2)

vii) Ultimate strength and space of connecting elements;

i. Wedge anchors

a. Ultimate shear strength; Ultimate shear stress of an anchor is obtained by the following equation.

$$ \tau_d = \text{minimum}\ (\sigma_{max}/\sqrt{3}\ ,\ 0.4\sqrt{E_{c1} \cdot F_{c1}}) \qquad (2.1.3) $$

where; σ_{max} : tensile strength of a wedge anchor

F_{c1} : specified compressive strength of existing concrete (kg/cm^2)

E_{c1} : modulus of elasticity of existing concrete (kg/cm^2)

Ultimate shear strength per wedge anchor q_d is

$$ q_d = \tau_d \cdot a_d \qquad (2.1.4) $$

where a_d is the shear section area per wedge anchor (cm^2)

When the screw part is involved in the shear section, the effective area is reduced to 80%.

b. Ultimate pull-out strength; Ultimate pull-out strength of an anchor is obtained by the following equation.

$$P_d = \text{minimum}\ [\sigma max \cdot a_d,$$
$$0.45\ \frac{\ell a}{Da} \cdot (\frac{\ell a}{Da} + 1) \cdot F_{c1} \cdot a_a] \qquad (2.1.5)$$

where ℓ_a, D_a and a_a are the length (cm), outside diameter (cm) and area (cm^2), respectively, of the embedded part of the anchor in the concrete of existing frame.

ii. Chipped cotters; Ultimate shear stress of a cotter is obtained by the following equation.

$$\tau_c = 0.20 \cdot F_{c2} \quad (kg/cm^2) \qquad (2.1.6)$$

where F_{c2} is the specified compressive strength of new concrete (kg/cm^2). The space of cotters is determined by the following equation.

$$\ell'_c = \ell_c \cdot F_{c2}/F_{c1} \qquad (2.1.7)$$

where ℓ_c is the length of a cotter (cm). Therefore, the pitch of cotters p_c is as follows.

$$p_c = \ell_c + \ell'_c \qquad (2.1.8)$$

iii. Adhesive cotters; Ultimate shear stress of a cotter (${}_c\tau_c$) is obtained by the following equation.

$${}_c\tau_c = \text{minimum}\ (0.25 \cdot F_{c1},\ 0.20 \cdot {}_cF_c) \qquad (2.1.9)$$

where, ${}_cF_c$ is the specified compressive strength of precast concrete cotters (kg/cm^2). The ultimate shear stress of a concrete cotter formed between adhesive cotters is obtained by Eq.(2.1.6) replacing F_{c2} with F_{c1}. The space of adhesive cotters is determined by the following equation.

$$\ell'_c = \ell_c \cdot \frac{{}_c\tau_c}{\tau_{c2}} \qquad (2.1.10)$$

where; ℓ_c : length of a precast cotter (cm)

τ_{c2} : ultimate shear stress of a placed concrete cotter (kg/cm^2)

The pitch of adhesive cotters is obtained by Eq.(2.1.8).

viii) Design of walls

i. The thickness of wall is determined so that the average shear stress of the wall (τ_w) may be less than or equal to 30 kg/cm^2 under the design shear force.

$$\tau_w = Q_w/(t_w \cdot L_w) \leqq 30.0 \qquad (2.1.11)$$

Q_w : design shear force (kg)

t_w : thickness of the wall (cm)

L_w : clear length of the wall (cm)

ii. For the shear stress τ_w above, the shear reinforcement ratio is calculated by the following equation.

$$p_w \geqq (\tau_w - \frac{F_{c2}}{20} / (0.5 \cdot \sigma_{wy}) \qquad (2.1.12)$$

where; p_w : shear reinforcement ratio of wall and

$0.0025 \leqq p_w \leqq 0.012)$

σ_{wy} : yield strength of shear reinforcement and 3000 kg/cm^2 may be taken for plain bars and the specified yield stress + 500 kg/cm^2 may be taken for deformed bars.

(3) Evaluation of The Seismic Safety

Evaluation of the seismic safety of the building after strengthening is based on Section 1.2 and 1.5.

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*1 See Fig.1.2.

*2 The type of construction in accordance with the connecting methods of additional walls and existing frames is shown in Fig.2.1.

*3 See Fig.2.1.1(a).

*4 See Fig.2.1.2.

*5 See Fig.2.1.1(b).

*6 See Fig.2.1.1(c).

*7 The idealized flow of lateral forces carried by each structural element and the connection is illustrated in Fig.2.1.3.

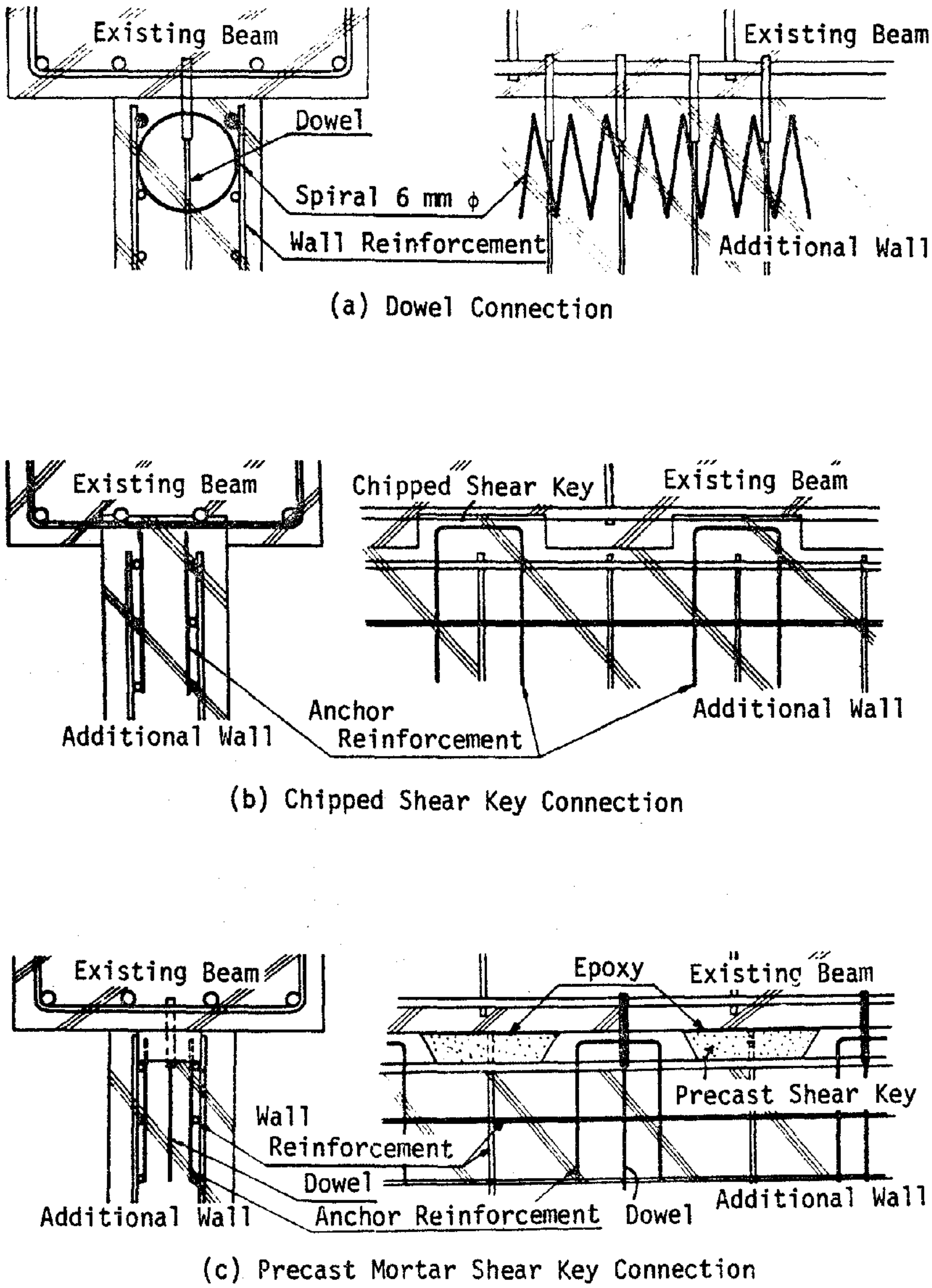

Fig. 2.1.1 Wall-Frame Connections

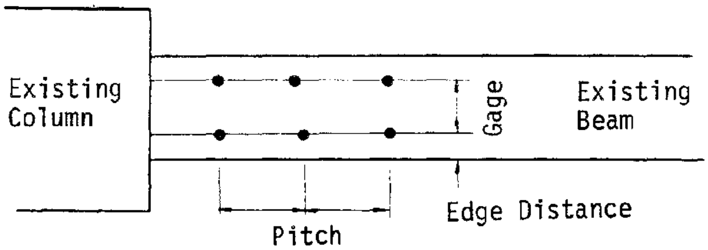

Fig. 2.1.2 Arrangement of Wedge Anchors

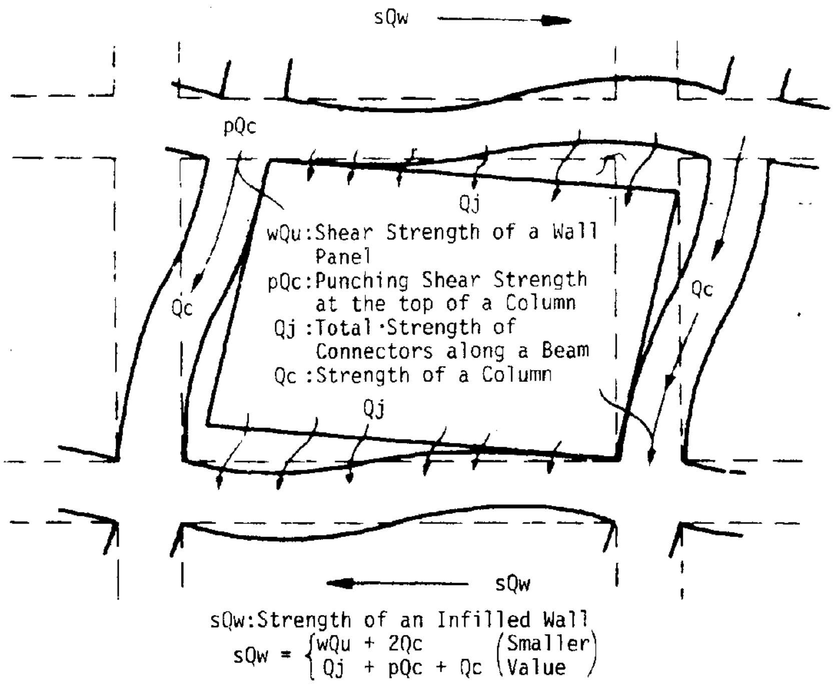

Fig. 2.1.3 Strength of an Infilled Wall

2.2 Strengthening by Wing Walls

2.2.1 General

This strengthening technique is applied to improve the strength of columns by placing slender wing walls which are not considered be shear walls so that the lateral force capacity of a building can be sufficiently increased. However, it must be taken into consideration during the design that the lateral force capacity of the building may be determined by the strength of existing beams even if the strength of columns is improved by using the technique above. Particicularly, this method should not be applied to the building in which the column span is narrow in order to avoid the shear failure of beams because additional wing walls considerably reduce the clear span of beams.

2.2.2 Aim of Seismic Capacity

(1) Aim of Capacity of a Total Building

The aims of earthquake resistance of a total building are already described in the section 1.2. In the case of strengthening of columns by wing walls, the following two methods may be applied for the previously described aims, that is, i) to improve the strength index C in order to make the building "strength resistant type", and ii) to form "beam yield type" mechanism by the strengthening in order to improve the ductility index F.

(2) Aim of Capacity of Columns with Wing Walls

In the cases of both "strength resistant type" and "ductility resistant type", the aim of strengthening by wing walls is to improve the strength of columns providing sufficient length and thickness of walls.

2.2.3 Plan of Strengthening

(1) Objective Buildings

This strengthening technique is applied to buildings in which shear failure type columns predominate and beams are strong enough, in another words, buildings of which lateral force capacity can be significantly improved when the strength of columns increase. In addition, the technique is also applied to buildings in which flexural failure type columns predominate, however, sufficient ductility is not expected, or to buildings in which excessive inelastic displacement is predicted even when the ductility of columns is improved, however, the earthquake resistance may be significantly improved when the failure mechanism is transformed into the "beam yield type".

The clear span of existing beams is reduced by wing walls. Hence, the technique is suitable generally for buildings having long span of columns because the flexural yielding should be expected to beams even under the reduced clear span.

(2) Strengthening Members

1) Because the aim of this method is to improve the strength of columns, the most columns of the building generally must be strengthened. Therefore, it is important to arrange wing walls so that structurally well-balanced both plan and elevation may be provided. The configuration of wing walls which may lead to extremely eccentric distribution of stiffness and/or strength of members in a story and/or between adjacent stories should be avoided.

2) When the beam yielding type mechanism is desired, the ratio of the clear span of a beam ℓ_0 to the depth of the beam D (ℓ_0/D) should be more than or equal to 4.0 (Fig. 2.2.1). However, this limitation may not be applied if further investigation confirms the flexural yielding of beams.

3) It is not desirable to apply this method to the captive columns of which the clear height is extremely short. An adequate investigation is

needed when applied.

2.2.4 Type of Construction and Structural Details

(1) Monolithically Cast Wing Walls

As shown in Fig. 2.2.2, it is possible to monolithically cast wing walls with the existing column after chipping a part of existing concrete and sufficiently arranging the lateral reinforcement of wing walls.

In this method, wing walls are eccentrically connected with the existing column since one side of the lateral reinforcement is arranged through the column as shown by Fig. 2.2.2. Hence, this method is possible only when beams are also eccentrically connected with the column.

Careful attention must be paid to the anchor of vertical and horizontal reinforcements of wing walls into existing concrete in order to avoid the out-of-plane deformation of walls. It is desirable to weld reinforcements of walls with existing transverse reinforcements. Attention must be also paid to the waterproofing at the connection when walls are used as external walls.

(2) Wing Walls Connected by Dowels

In this construction method, the existing column and wing walls are connected by dowels with wedge anchors or by other type of dowels, and shear forces are transmitted by the dowels. As shown by Fig. 2.2.3, wing walls are formed by cast-in-place concrete or by placing precast concrete grouting the connection.

It should be considered during the design that the structural behavior of the column with wing walls constructed by this method may be considerably different from that of the other type of column because the connection is not so monolithic as the other type of connection described in (1).

Dowels should be placed within concrete core of beams and columns, and it is desirable to place walls so that the centers of walls and a column will be consistent.

(3) Structural Details

It is desirable to accomplish the construction of strengthening by wing walls in accordance with the following requirements or recommendations.

1) Wing walls shall be in principle symmetrically placed at both sides of the column.

2) In the method described in Section 2.2.4 (1), the length of a wing wall on one side L shall be more than 1/2 the depth of the column D and more than 50 cm, and the thickness of the wall t shall be more than 1/3 the width of the column b and more than 20 cm.

3) In the method described in Section 2.2.4 (2), the length of the wing wall on one side L shall be more than 80 cm, the ratio of L to the clear height h_o, L/h_o, shall be more than 1/3, and the wall thickness t shall be more than 15 cm.

4) Vertical and lateral reinforcement ratios of wing walls, p_{sv} and p_{sh}, respectively, shall be more than 0.25%.

5) The pitch (the interval along the direction to the height and length of the wall) of wedge anchors shall be more than 7.5 times the outside diameter of the bolt D_d and less than 30 cm, and the gage (the interval along the direction to the thickness) shall be more than 5.0 times D_d.

6) The embedded length of a wedge anchor bolt shall be more than 5.0 times the outside diameter at the shear face and more than the covering thickness.

7) Anchor reinforcement shall be adequately arranged around the dowel reinforcement of wing walls in order to prevent splitting of concrete.

8) The depth of cover concrete of wing walls shall follow "Standard for Design of Reinforced Concrete Structure" of AIJ, and in case of the method mentioned in Section 2.2.4(1), the chipped part of the

existing column is desirable to be thickened as shown in Fig. 2.2.2.

9) In the method described in Section 2.2.4 (1), the end of the vertical reinforcements near by the outside of wing walls shall be adequately welded with existing stirrups, and the non-spliced lateral reinforcements shall also be welded with existing hoops at the interval less than or equal to 50 cm.

2.2.5 Design Calculations

(1) Process of Calculations

The process of calculations for the design of strengthening by the wing walls is as follows.

1) To establish the aim of strengthening referring the result of the evaluation of seismic safety.

2) To select the construction method of wing walls and to determine the structural details.

3) To calculate the ultimate strength of columns with wing walls and beams connected by columns.

4) To compute E_0-index of the strengthened building based on the third evaluation method of "Criterion on the Evaluation of Seismic Safety" considering the failure type of the strengthened frame.

5) To calculate the seismic index of the structure ${}_RI_S$, and to judge whether or not the aim of strengthening is satisfied. When not satisfied, to recalculate returning to the process 3) after increasing the strengthened part or changing the method of construction and structural details.

(2) Seismic Capacity of Strengthened Members

1) Columns with monolithically cast wing walls *1

The ultimate strength of the column with additional wing walls mentioned in Section 2.2.4 (1) may be smaller value of the following shear force at the ultimate flexural strength Q_{Mu} or the ultimate shear strength Q_{su}.*2

$$Q_{Mu} = \phi \cdot 2M_u/h_o \tag{2.2.1}$$

$$M_u = (0.9 + \beta).a_t.\sigma_y.D + 0.5N.D \left\{1 + 2\beta - \frac{N}{\alpha e.b.D.F_{c1}}\left(\frac{a_t.\sigma_y}{N} + 1\right)^2\right\} \tag{2.2.2}$$

where, $\alpha_e = (1 + 2\alpha.\beta)/(1 + 2\beta)$

α and β are shown in Fig. 2.2.4

ϕ : reduction coefficient (= 0.8)

h_o : clear span of the column (cm)

a_t : gross area of tension reinforcements of the column (cm^2)

σ_y : yield stress of longitudinal reinforcement of the column (kg/cm^2)

N : axial force of the column (kg)

F_{c1} : specified compressive strength of existing concrete (kg/cm^2)

b : width of the column (cm)

D : depth of the column (cm)

$$Q_{Su} = \phi \left[0.8\sqrt{F_{c1}}\left(\frac{\ell_w}{h_o}\Sigma A + 0.5\left\{P_w.\sigma_{wy} + P_{sh}\frac{t(\ell w-D)}{b.D}\right\}b.D\right) + 0.1N\right] \tag{2.2.3}$$

where, ℓ_w : total depth of the column with wing walls (Fig. 2.2.4) (cm)

ΣA : total sectional area of the column with wing walls (cm^2)

$P_w.\sigma_{wy}$: product of the steel ratio and the yield stress of the transverse reinforcement of the column (kg/cm^2)

$P_{sh}.\sigma_{sy}$: product of the steel ratio and the yield stress of the lateral reinforcement of the wing wall (kg/cm^2)

t : thickness of the wing wall (cm)

2) Columns with wing walls connected by dowels[*3]

i) The ultimate strength Q_u of the column with wing walls described in Section 2.2.4 (2) is calculated by Eq.(2.2.4) assuming the lateral shear force Q_T carried by the inclined compression members, which are idealized as the model of wing walls, and the lateral shear force Q_c carried by the existing column.[*4]

$$Q_u = Q_T + Q_c \tag{2.2.4}$$

ii) The lateral shear force Q_T carried by the truss model takes the minimum value among the lateral force Q_{T1} at the ultimate compressive strength of the inclined members, the ultimate shear strength of the connection Q_{T2} at the top and bottom of the wing walls, and the ultimate shear strength of wing walls Q_{T3}, which are obtained by the following equations (2.2.5) through (2.2.7).

$$Q_{T1} = 2\alpha_B . t^2 . f_c (L_1/L_2) \leqq 2(N + a_g . \sigma_y)(L_1/H) \tag{2.2.5}$$

$$Q_{T2} = \text{smaller value of the following two}$$

$$\begin{cases} (\Sigma A_d \cdot \sigma_{max} / \sqrt{3}) + 0.25\alpha_B . t^2 . f_c (H/L_2) \\ (0.4\ \Sigma A_d \sqrt{E_c . F_{c1}}) + 0.25\alpha_B . t^2 . f_c (H/L_2) \end{cases} \tag{2.2.6}$$

$$Q_{T3} = \Sigma A_w (f_s + 0.5 P_{sh} \cdot \sigma_{sy}) \tag{2.2.7}$$

where, α_B : effective width ratio of the inclined compression member and may be 2.0, except when it is determined by a special investigation.

$a_g \cdot \sigma_y$: product of the gross area and the yield strength of the longitudinal reinforcement of the column (kg)

H : story height (cm)

E_c : modulus of elasticity of the existing concrete (kg/cm^2)

$f_c = 0.85\ F_{c1}$ (kg/cm^2)

ΣA_d : sum of the sectional areas of wedge anchor bolts placed at the horizontal connection of wing walls (sum of the area in walls at both sides) (cm^2)

σ_{max} : tensile strength of a wedge anchor bolt (kg/cm^2)

ΣA_w : horizontal sectional area of the wing walls at both sides (cm^2)

$P_{sh} \cdot \sigma_{sy}$: product of the steel ratio and the yield stress of the horizontal reinforcement, where $P_{sh} \leqq 1.2\%$.

f_s : allowable shear stress of concrete of the wing wall (kg/cm^2) ; it is based on "Standard for Design of Reinforced Concrete Structures" of AIJ.

iii) The shear force Q_c carried by the existing column is obtained by the following equation.

$$Q_c = \text{minimum}\ (\alpha_1 \cdot Q_{Mu}\ ,\ \alpha_2 \cdot Q_{Su}) \qquad (2.2.8)$$

where, Q_{Mu} and Q_{Su} are the shear force at the ultimate flexural strength and the ultimate shear strength, respectively, of the existing column, and they are calculated by the equations used in the third evaluation method of "Criterion on the Seismic Safety". However, the axial force N used for the calculation of Q_{Mu} and Q_{Su} may be $N = N(\text{long-time}) - \frac{Q_T}{2}(\frac{H}{L_1})$, or zero for $N < 0$. The symbols α_1 and α_2 are the reduction coefficient of the shear force of the existing column at the failure of the wing walls. They may be respectively $\alpha_1 = 0.7$ and $\alpha_2 = 1.0$ considering the compatibility of the deflection.

3) Ultimate strength of existing beams

The ultimate strength of existing beams is calculated by the equations in the third evaluation method of "Criterion on The Evaluation of Seismic Safety".

4) Ductility index of the column with wing walls

The ductility index F of the column with wing walls may be 1.0. However, when the beam yielding type mechanism is formed after the construction of wing walls, F-index may be 3.0 for columns with wing walls.

(3) Evaluation of the Seismic Safety

The seismic safety of a strengthened building is evaluated based on Section 1.2 and 1.5.

---

*1 The dimension of a column with monolithically cast wing walls is shown in Fig.2.2.4.

*2 The calculated ultimate strengths by the proposed equations are compared in Fig.2.2.6 with experimental results.

*3 The dimension of a column with wing walls connected by dowels is shown in Figs.2.2.3 and 2.2.5.

*4 The analytical model is illustrated in Fig.2.2.5, and the calculated ultimate strengths by the proposed equations are compared in Fig.2.2.7 with experimental results.

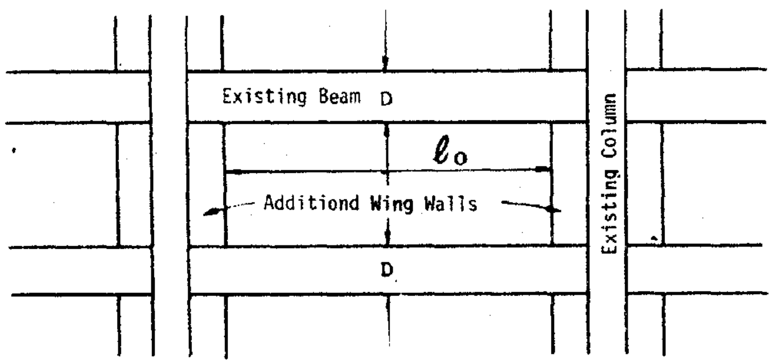

Fig. 2.2.1

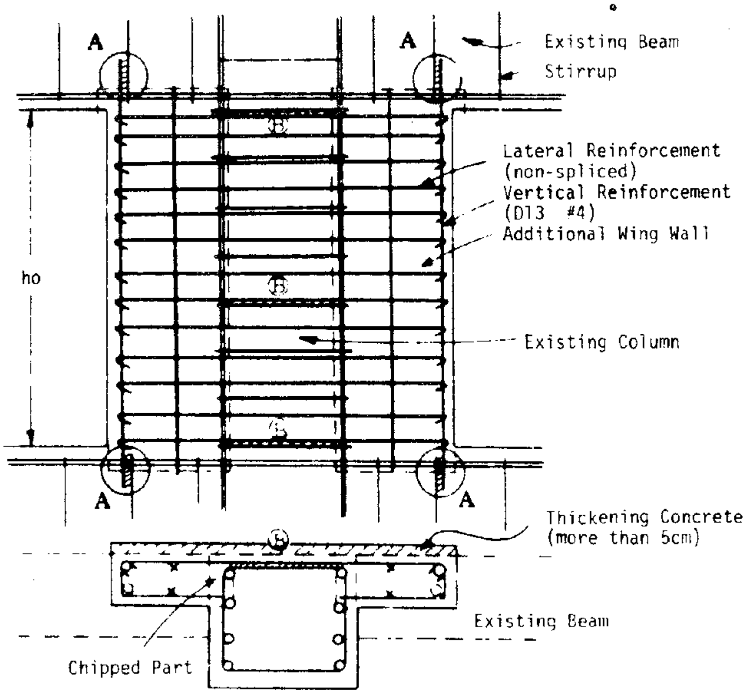

Fig. 2.2.2 Monolithically Cast Wing Walls

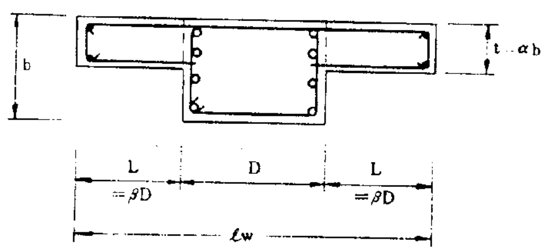

Fig. 2.2.4

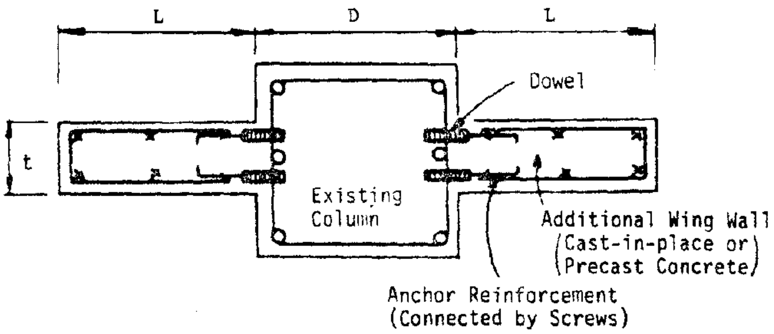

Fig. 2.2.3 Wing Walls with Dowel Connections

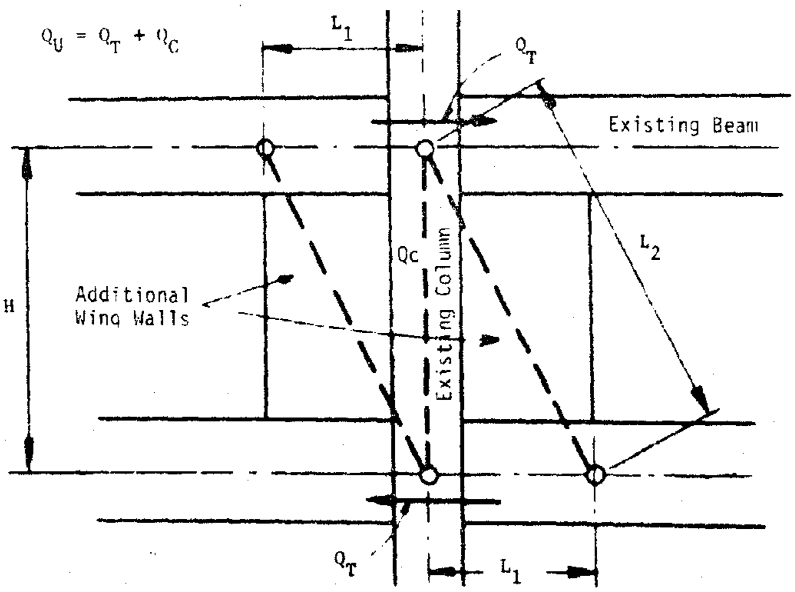

Fig. 2.2.5 Strength of a Column with Wing Walls Connected by Dowels

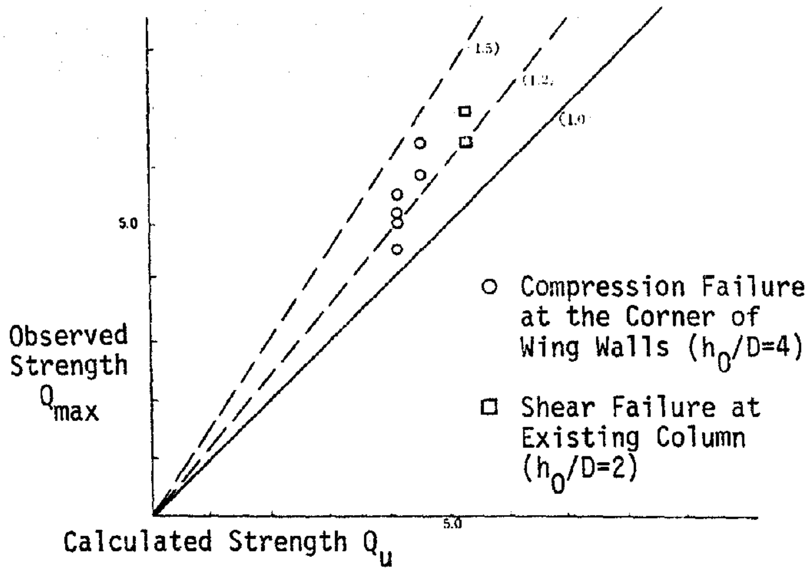

Fig. 2.2.7 Ultimate Strength of Columns with Wing Walls Connected by Dowels

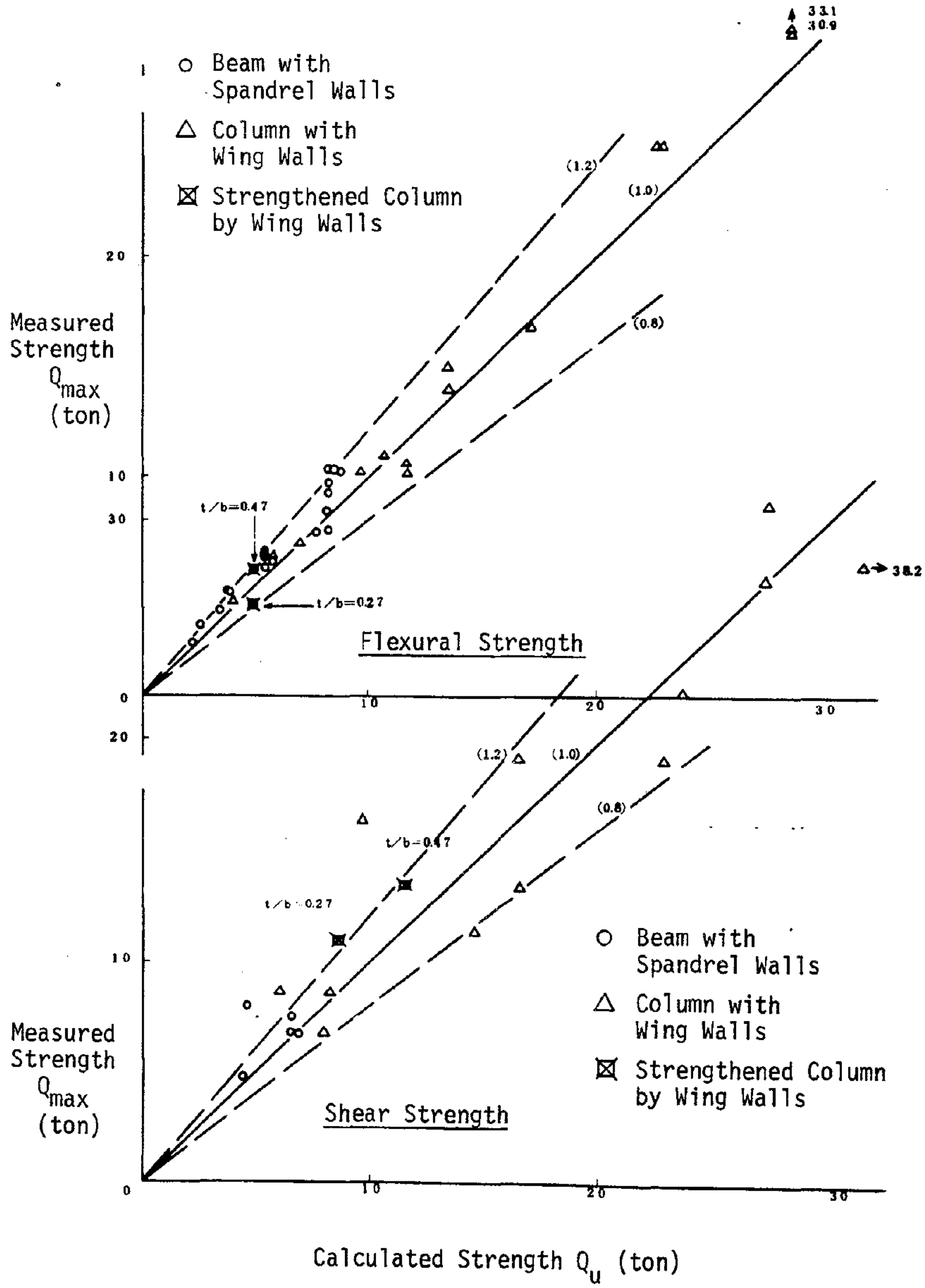

Fig. 2.2.6 Ultimate Strength of Columns with Monolithically Cast Wing Walls

2.3 Strengthening of Columns

2.3.1 General

The aim of this strengthening is to improve the earthquake resistance of a building upgrading the seismic capacity of columns by means of one of the following methods, or by their combinations, that is, to improve the ductility of columns avoiding shear failure, to equalize the stiffness of columns, and to increase the flexural capacity of columns. Although it is desirable to strengthen all the columns which lack the sufficient ductility or in which the stiffnesses are considerably unequal at the story under consideration, it should be recognized during the design that there is a limit to increasing the ductility of columns.

2.3.2 Aim of Seismic Capacity

(1) Capacity of Strengthened Columns

The aim of the strengthening is to make columns ductile, that is, to increase the ductility index F. Even in the case of strengthening to increase the flexural strength, the aim is to increase the index F as well as the strength index C.

(2) Capacity of a Total Building

The flexural failure precedence type is aimed as the earthquake resistance of a strengthened building. In another words, the earthquake resistance of the building is upgraded improving the ductility index F by preceding the flexural failure of columns.

2.3.3 Plan of Strengthening

(1) Objective Buildings

The buildings to which this strengthening technique may be applied are classified into the following three groups when both the strength and the ductility of beams are sufficient.

1) Those having few amount of shear walls, in which the ultimate shear

strength of columns is lower than the ultimate flexural strength. The strengthening method to improve the ductility of columns is effective to such buildings.

2) Those in which the distribution of lateral forces carried by columns is significantly uneven due to the existence of spandrel walls. The method to equalize the stiffness of columns is effective.

3) Those havings few shear walls of which the lateral force capacity is considerably low while the ductility is sufficient. The method to increase the flexural strength of columns is effective.

(2) Location of Strengthening

Although it is desirable to strengthen all the columns which lack the sufficient ductility at the story under the consideration, it should be recognized that there is a limit to improving the ductility of columns.

2.3.4 Type of Construction and Structural Details

(1) Strengthening to Improve the Ductility of Columns*1

1) Type of Construction

i) Method to increase the column size adding additional reinforcement of welded wire fabrics adjacent to the existing column.

ii) Method to increase the column size adding additional reinforcement of welded ties adjacent to the existing column.

iii) Method to encase the existing column with rectangular or circular steel sections.

iv) Method to encase the existing column with steel straps.

In any cases above, any voids are grouted to fill with concrete or mortar.

2) Structural Details

i) When the aim of strengthening is to improve the ductility alone, any types of construction method described above are accomplished by providing gaps of 3 cm at both the top and bottom of the column.

ii) In the case of strengthening by additional reinforcement of welded wire fabrics, the type of the lapped splice in  -shape is easier in the construction than that in  -shape. The lapped length measured by that between outermost crossing wires of each fabric sheet shall be more than the space of crossing wires plus 10 cm, and more than 20 cm.*2

iii) In the case of strengthening by steel straps, both the space and the width of straps are desired to be around 10 cm.

iv) In the case of strengthening by steel sections or steel straps, the thickness of steel shall be more than 3.2 mm.

(2) Strengthening to Equalize the Stiffness of Columns

1) Type of Construction

i) Removal or separation of spandrel walls

2) Structural Details

i) It is desirable to apply this technique together with that described in (1) above, when the shear failure of columns is still expected even after the increased clear span of columns.

ii) The gap between columns and spandrel walls shall be greater than 3 cm. The separated spandrel walls shall be safe enough against the out-of-plane deformation.

(3) Strengthening to Increase the Flexural Capacity of Columns

1) Type of Construction

i) Enlargement of the column size

2) Structural Details

i) The additional longitudinal reinforcement which is taken into account as a part of the flexural reinforcement of the column should be arranged penetrating the slab so that it may be sufficiently anchored.

ii) The additional shear reinforcement shall be adequately arranged against the increased flexural capacity of the column.

2.3.5 Design Calculations

(1) Calculation Procedure

The design calculations for the strengthening of columns are accomplished along the following procedure.

1) To select appropriate construction method referring the result of the evaluation of seismic safety, where the strength of beams should be sufficiently discussed. The calculation process for each type of construction method is separately described in the following items 2) through 4).

2) Calculation process for the strengthening to improve the ductility

i) To discuss whether or not the strengthening is possible on the basis of Eq. (2.3.5) shown later on.

ii) To approximate the required ductility index F.

iii) To calculate the required ductility factor μ corresponding to the F index.

iv) To determine the required shear reinforcement ratio P_{w2} corresponding to the obtained ductility index μ.

v) To judge whether or not the aim of the strengthening is satisfied. The process is completed if the strengthening is feasible as well as the aim is satisfied. Otherwise, recalculation is needed returning to the step ii).

3) Calculation process for the strengthening to equalize the stiffness

i) To assume the clear height of columns after the removal or separation of spandrel walls.

ii) To obtain the ductility and strength indices F and C, respectively, by computing both the flexural and shear strengths of columns.

iii) To judge whether or not the indices F and C are satisfactory to the aim of the strengthening, and to discuss, if not satisfactory, the combination with another type of construction.

4) Calculation process for the strengthening to increase the flexural strength

i) To approximate the aimed values of F and C indices.

ii) To determine the amount of flexural reinforcement corresponding to the flexural strength of columns which satisfy the aimed C index.

iii) To determine the shear reinforcement ratio P_{w2} corresponding to the aimed F index.

iv) To judge whether or not the aim of strengthening is satisfied. The calculation is completed if the strengthening is feasible as well as the aim is satisfied. Otherwise, the process is repeated returning to the step i).

(2) Seismic Capacity of Strengthened Column

The seismic capacity of strengthened columns is evaluated by the following items 1) and 2), when the limitations described in item 3) are satisfied.*3

1) Ultimate flexural strength of columns

The ultimate flexural strength of columns is calculated by the Eq. (10) of "Criterion on the Evaluation of Seismic Safety" shown as below.

$$M_u = 0.8a_t.\sigma_y.D + 0.5N.D(1 - \frac{N}{b.D.F_{c1}}) \qquad (2.3.1)$$

$$Q_{Mu} = \alpha.M_u/h_o \qquad (2.3.2)$$

where, a_t : sectional area of tension reinforcement (cm^2)

σ_y : yield stress of flexural reinforcement (kg/cm^2), and may be 3000 kg/cm^2 for plain bars and the specified yield strength + 500 kg/cm^2 for deformed bars.

b : width of the existing column (cm)

D : depth of the existing column (cm)

N : axial force of the existing column (kg), and shall be

$0 \leq N \leq 0.4 b.D.F_{c1}$

F_{c1} : compressive strength of existing concrete (kg/cm^2)

h_o : clear height (cm)

α : α is in principle obtained by a detailed calculation, however, it may be 2.0 for approximation.

The strength of columns with additional flexural reinforcement is calculated by the following equation (Fig. 2.3.1).

$$M_u = a_t.\sigma_y.g + a_{t2}.\sigma_{y2}.g_2 + 0.5N.D_2(1 - \frac{N}{b_2.D_2.F_{c1}}) \qquad (2.3.3)$$

where, g : distance between the centroids of tension and compression reinforcements of the existing column (cm)

g_2 : similar distance to that above, but for additional flexural reinforcements (cm)

a_{t2} : sectional area of additional tension reinforcement (cm^2)

σ_{y2} : yield stress of the additional reinforcement (kg/cm^2), and may be 3000 kg/cm^2 for plain bars and the specified yield strength + 500 kg/cm^2 for deformed bars

b_2 : width of the column after strengthening (cm)

D_2 : depth of the column after strengthening (cm)

2) Ultimate shear strength of columns

The ultimate shear strength of columns is calculated by Eq. (13) of "Criterion on the Evaluation of Seismic Safety" shown as below.

$$Q_{Su} = \left\{ \frac{0.053 P_{t2}^{0.23}(180 + F_{c1})}{\frac{M}{Q.d_2} + 0.12} + 2.7\sqrt{P_w.\sigma_{wy} + P_{w2}\sigma_{wy2}} + 0.1.\frac{N}{b_2.D_2} \right\} 0.8 b_2.D_2 \qquad (2.3.4)$$

where, $1 \leq M/(Q \cdot d_2) \leq 3$, and

P_{t2} : tension reinforcement ratio to the area of the increased column section (%)

P_w : shear reinforcement ratio for existing reinforcement to the increased column section

P_{w2} : shear reinforcement ratio for additional reinforcement to the increased column section, and the value of $(P_w + P_{w2})$ shall be 0.012 when the sum exceeds 0.012.

σ_{wy} : yield stress of the existing shear reinforcement (kg/cm^2)

σ_{wy2} : yield stress of the additional shear reinforcement, (kg/cm^2) and σ_{wy} and σ_{wy2} may be 3000 kg/cm^2 for plain bars and the specified yield strength + 500 kg/cm^2 for deformed bars

d_2 : effective depth of the increased column section (cm)

M/Q : it is obtained, in principle, by detailed calculation, however, it may be $h_{o/2}$ for the approximation

3) Structural limitations for desirable ductility

The columns which require high value of F-index should satisfy the following limitations.

$$70\, P_t + \sigma_o \leq 37.5\, h_o/D \qquad (2.3.5)$$

where, P_t : tension reinforcement ratio (%)

σ_o : axial stress (kg/cm^2)

h_o : clear height (cm)

D : depth of the column (cm)

(3) Seismic Capacity of Strengthened Building

The seismic capacity of the strengthened building is evaluated as described in Section 1.2 and 1.5.

*1 See Fig.1.3 for construction methods.

*2 The lapped length of welded wire fabrics is illustrated in Fig.2.3.2.

*3 Experimental results of the ultimate strength of strengthened columns are shown in Fig.2.3.3, and the increased ductility factors of strengthened columns are shown in Fig.2.3.4.

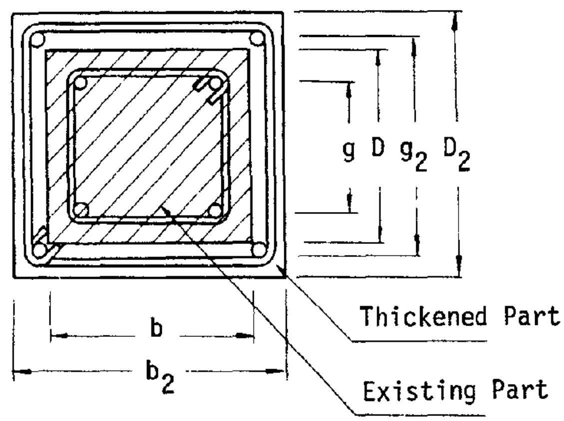

Fig. 2.3.1

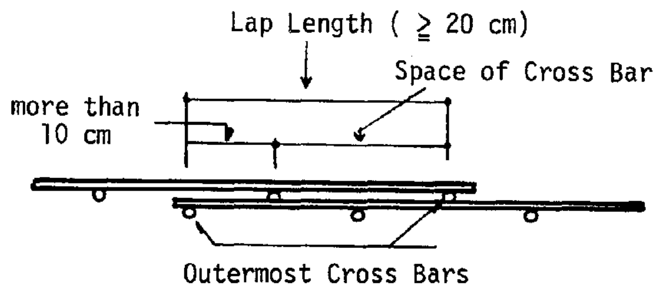

Fig. 2.3.2 Lap Length of Welded Wire Fabrics

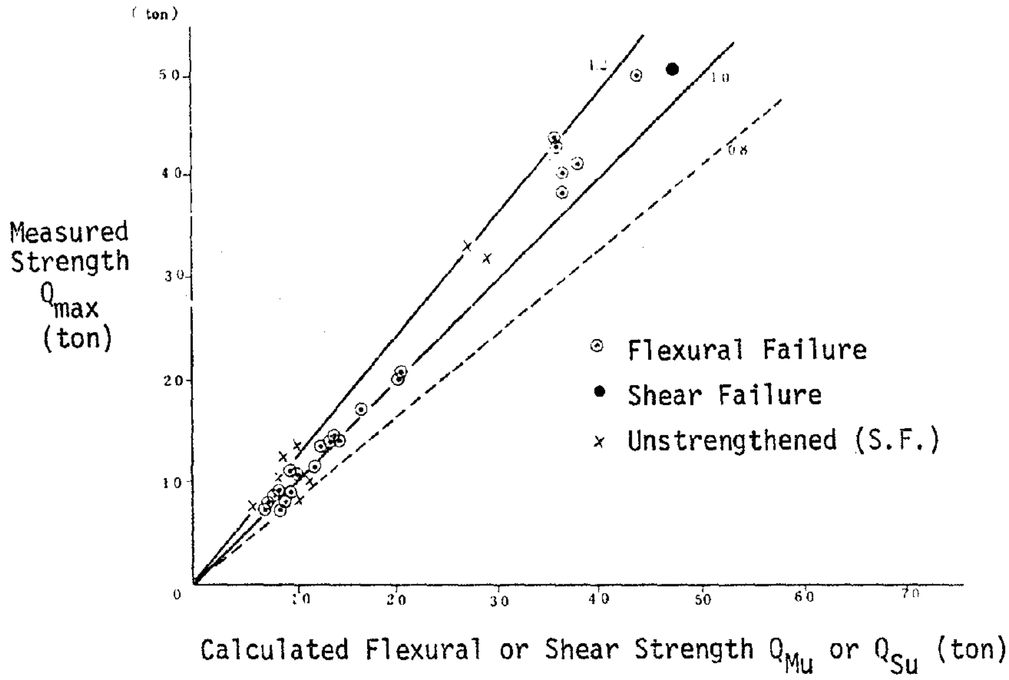

Fig. 2.3.3 Ultimate Strength of Columns

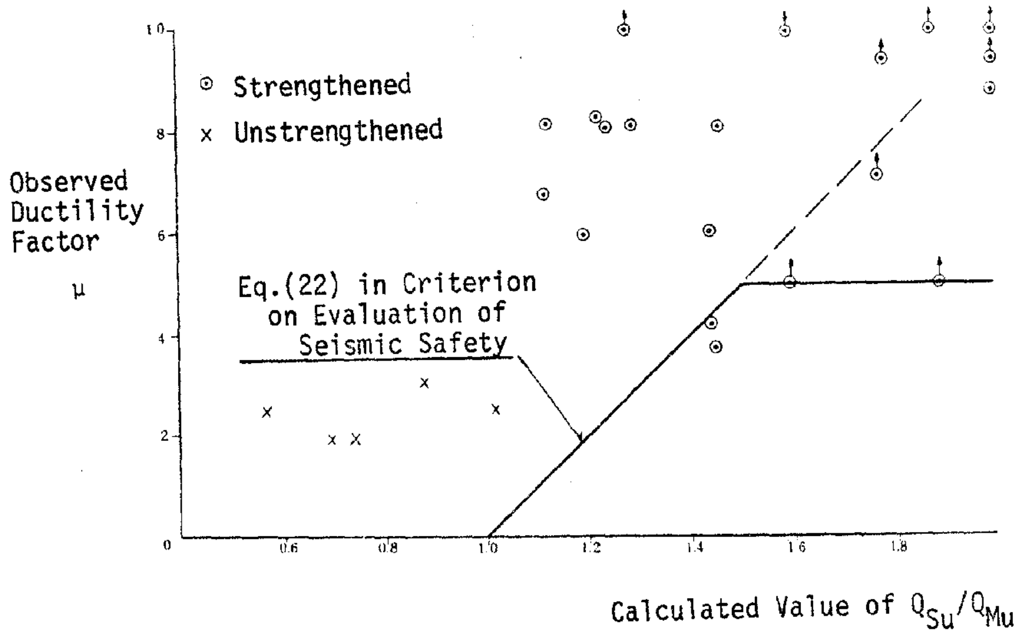

Fig. 2.3.4 Ductility of Columns

2.4 Other Strengthening Methods

2.4.1 General

In this section, basic matters for strengthening methods by buttresses, braces, additional columns and others are described.

2.4.2 Method by Buttresses

(1) General

The objective of this strengthening method is mainly to increase the lateral force capacity of the building by providing buttresses outside the building.

(2) Aim of Seismic Capacity

For the strengthening by buttresses, the aim of the capacity should be clearly established. In addition, it should be investigated that each part of the buttress, the connection between the buttress and the existing building, and the foundation have adequate strengths.

(3) Plan of Strengthening

This strengthening method is beneficial to buildings of which the overturning strength and the lateral force capacity are low and which have sufficient free ground around them. Buttresses are in principle connected with the structural frames of the building, and shall be provided in good distribution through the stories on both sides of the building.

(4) Construction and Structural Details

1) When this method is applied, the following items must be investigated.

i) Ultimate resistant moment considering the ground or piles.

ii) Stress in each portion of the additional footings under the above mentioned moment.

iii) Flexural and shear strengths of buttresses.

iv) Strength and details of connections for buttresses and existing columns.

2) The construction is desired to follow the structural details shown below.

i) Buttresses have columns and beams at their periphery and at the floor level respectively.

ii) The longitudinal bars, at least corner bars, of beams of the battress are adequately welded with the longitudinal bars of the existing beams.

iii) The foundation of the buttress has continuous footings which have adequate size of the section.

2.4.3 Method by Braces

(1) General

The objective of this strengthening method is mainly to increase the strength capacity of the building adequately providing braces within structurally important existing frames.

(2) Aim of Seismic Capacity

Even if the cross-X type braces are used, the compressive braces alone shall be in principle considered effective. Braces shall be adequately provided so as to avoid significant deterioration of structural property of existing structures, mainly of beam-column connections.

(3) Plan of Strengthening

This strengthening method is beneficial to buildings of which the beam-column joints have adequate strength and in which appropriate arrangement of braces is possible.

In this method, the arrangement of braces shall be well balanced, and especially, the smooth transmission of stress from the upper story to the lower story should be designed considering the distribution of the rigidity in the building.

(4) Construction and Structural Details

1) When this method is applied, the following items should be investigated.

i) Compressive strength and buckling strength of braces.

ii) Additional stress of the main structural members and the foundation of frames where braces are arranged.

iii) Beam-column joints connecting with braces.

2) The construction is desired to follow the structural details shown below.

i) Braces should be arranged so that their center lines pass through centers of beam-column joints.

ii) The ends of braces are designed and constructed so that they are connected with the existing frame at their faces.

iii) The connections between braces and existing portions are designed so as to transmit the out-of-plane shear force corresponding to the weight of braces.

iv) In the case of compressive braces of concrete, the longitudinal reinforcement ratio shall be more than 0.8% and the shear reinforcement ratio shall be more than 0.2%.

v) Braces should be arranged so continuously that the transmission of stresses through the stories may be smooth.

2.4.4 Other Methods

In other types of strengthening such as that by additional columns, that for beams, and that for the improvement of stiffness distribution, the effect of strengthening shall be in principle verified on the basis of experiments.

2.5 Strengthening of Foundations

2.5.1 General

It is desirable that the plan of strengthening is established so as not to need the strengthening of foundations. Foundations may be strengthened in principle only in the cases where the strengthening can be accomplished by convenient methods, the construction is technically and economically feasible, and significantly improved earthquake resistance of the building due to the strengthening is possible.

2.5.2 Aim of Strengthening

(1) The aim of strengthening of foundations is to make the strengthening of the upper structure effective so that the required seismic capacity to the total structure can be satisfied.

(2) Foundations must be able to support the long-term loads of structures after the strengthening.

(3) When it is presumed that the subsidence of the ground, the negative friction of piles or the liquefaction of sand layer during an earthquake may occur and result in undesirable effects on the structural capacity of the building, such effects should be avoided by improving the ground based on a proper construction method.

2.5.3 Estimation of Bearing Strength and Subsidence

(1) The bearing strength of the ground and piles, the subsidence of the ground, the negative friction, and the lateral force resistance of piles are calculated in accordance with the "Standard for Design of Foundation Structures" of AIJ.

(2) The bearing strength of the ground and piles after the strengthening shall be in principle the same as those in general case. The allowable bearing strength against seismic loads, however, may be the ultimate bearing strength.

2.5.4 Estimation of Bearing Strength of Strengthened Foundations

The bearing strength of additional foundations may be added to that of the existing foundation except special cases.

2.5.5 Structural Details

(1) Additional foundations should not be constructed eccentrically in principle.

(2) Foundations should not be used, in principle, together with those of defferent type.

(3) Connections between additional foundations and existing portions should be constructed so that as closer strength and stiffness as possible to those of monolithic construction can be obtained.

(4) The construction of additional foundations should be performed so as not to harm existing foundations.

(5) In the selection of construction methods for additional foundations, the safety during the construction and the feasibility shall be sufficiently investigated.

2.6 Strengthening and Repair of Non-Structural Elements

2.6.1 General

The aim of strengthening and repair of non-structural elements is to prevent the separation and fall of such elements as external finish materials during an earthquake. The matters and methods described herein are concerned with only external walls and, in addition, concerned with the security of human life associated with the cases in which the separation and fall of elements may directly injure the people, and shut up the escape passages.

2.6.2 Aim of Capacity

(1) Capacity of Elements

The most essential aim of repair and/or strengthening of existing non-structural elements by appropriate methods is to prevent the fall of such elements during an earthquake, in order to secure the human life. The aim of the capacity of elements depends on the site condition of the building, the type of structural system, the characteristics of materials, and others.

(2) Capacity of a Total Building

The structural capacity of the building after the repair and/or strengthening should not be different from that before the retrofitting. When the structural capacity of the building may be changed by the retrofitting, the investigation from several view points should be needed.

2.6.3 Plan of Retrofitting

(1) Elements which Need Retrofitting

Non-structural elements which may need retrofitting are those shown in the following items.

1) External walls of concrete blocks and glass blocks, and curtain walls.

2) Window frames and glasses of external walls.

3) External finishing materials such as stones and tiles.

4) Signboards and lighting equipments on external walls.

Although comparatively large lamps on the roof floor are important, they are out of the object herein.

2.6.4 Retrofitting Methods

The retrofitting methods which can increase the value of seismic index of non-structural elements I_N calculated in the evaluation of seismic safety of the building may be selected in principle. The methods are shown as follows.

(1) External Walls, Openings of External Walls and External Finishing Materials

1) To avoid dangerous conditions of the fall of materials by improving the deflection capacity replacing materials of external walls and finishings.

2) When the same materials as those before the retorfitting are used, the methods in which the deflection capacity or the one body condition with the base members are considered should be selected.

3) To provide stoppers for falling elements such as eaves.

(2) Signboards and Lighting Equipments on External Walls

1) To remove signboards and lighting equipments.

2) To reconstruct the connection of signboards and lighting equipments.

3. GUIDELINE FOR CONSTRUCTION OF RETROFITTING

3.1 General

(1) Scope

This chapter is applied to the construction of strengthening methods described in the previous Sections 2.1 through 2.5. The matters not mentioned herein should follow the standard specification "JASS" of AIJ.

(2) Plan of Construction

The plan of construction should be established so that the effect of retrofitting expected during the design can surely be actualized. In the plan of construction, careful attention should be paid understanding the use condition of the building so that disturbances associated with the noise, dust and contamination during the construction can be minimized and so that the safety of users of the building and the safety during the construction can be secured.

3.2 Materials

(1) Materials for Mortar and Concrete

1) Cement

The following types of cement may be used, that is, normal, high-early-strength or extremely high-early-strength portland cement specified in JIS R 5210 (portland cement), or A type cement specified in JIS R 5211 (blast-furnace-slug cement), JIS R 5212 (silica cement) or JIS R 5213 (fly-ash cement).

2) Aggregate

Sand, gravel or gravel pebble shall be used for aggregate. The nominal maximum size of the coarse aggregate may be specified in accordance with the placing portion. The fine aggregate used for the mortar for the strengthening of columns shall be that of I or II class specified in JASS 5.3.3.

3) Air-entraining admixture

Air-entraining admixture or Air-entraining water reducing admixture is generally used.

4) Admixture

Only for the case with normal portland cement, fly-ash may be used when necessary. The fly-ash, however, shall be that specified in JIS A 6201 (fly-ash), and the dosage shall be less than that for A type cement specified in JIS R 5213 (fly-ash cement).

5) Expansive admixture

Expansive admixture may be used when required.

(2) Materials for Grout Mortar

1) Cement

Cement described in Section 3.2 (1) 1) is used.

2) Aggregate

Fine aggregate of I or II class specified in JASS 5.3.3 is used.

3) Admixture

Surface active agent for concrete may be used as the admixture.

4) Expansive admixture

Expansive admixture must be used for grout mortar. The foamy material, however, such as aluminum powder must not be used.

(3) Reinforcement

Reinforcing bars shall be in principle the standard materials specified in JIS G 3112 (steel bar for concrete reinforcement) and shall be normally deformed bars. Welded wire fabrics shall conform to the standard in JIS G 3551 (welded wire fabric), and the diameter of wire shall be more than 4 mm.

Wedge anchors of which the quality and capacity are adequately verified shall be used, and the nominal effective shear diameter shall be 13 mm through 22 mm.

(4) Steel

Steel shall conform to the specification JIS G 3101 (general structural rolled steel) and the thickness of steel plate shall be not less than 3.2 mm.

3.3 Removal of Finishing Materials and Chipping of Concrete

(1) Removal of Existing Finishing Materials

Before retrofitting, interior finishing materials and the finishing materials on concrete members such as plaster and mortar are removed.

(2) Treatment and Chipping of Existing Concrete

The connecting surface of concrete with new mortal or concrete are properly roughened or chipped. For the case with chipped shear cotters, the constructed dimension shall be as close to that shown by the design drawing as possible and shall not be less than that designed. The chipped surface shall be as even as possible. In addition, the retrofitting method in which the chipping work is minimized should be selected.

3.4 Reinforcement Work

(1) In the construction to place new reinforced concrete members, new reinforcing bars shall be anchored into the existing members or into their main reinforcements by effective methods.

(2) When new reinforcements are anchored to existing members by means of wedge anchors, the space of anchors shall be more than 7.5 times their effective diameter at the shear face along the direction of shear force, and more than 5 times the diameter along the perpendicular direction to the shear force.

Wedge anchors shall be provided more than 10 cm apart from the end of concrete, and shall be provided between longitudinal bars. The embedded length of anchors shall be more than 5 times their effect diameter of the shear face and more than the covering thickness.

(3) When additional reinforcements are anchored with reinforcements of existing frame, new reinforcements shall be hooked with the bent more than 135° or welded with existing reinforcements.

(4) In the case of strengthening by welded wire fabrics for improving the ductility capacity of columns, the length of the lapped splice of welded wire fabrics which is measured by the distance between outermost crossing wires of each fabric sheet shall be more than the space of the crossing wires plus 10 cm and more than 20 cm.

(5) In the case of anchorage of new reinforcements by welding with the existing main reinforcements, careful attention must be given considering the welding ability of steel bars so that the mechanical characteristics of the reinforcements may not be changed by welding. The welding by different posture shall be performed by experienced welders who have licensed for each welding posture specified in JIS Z 3801 (testing method and judgement criterion for the examination of welding technics) or by those who have qualified by the Japan Welding Institute or by other associations.

3.5 Concrete Work

(1) Plan of Concrete Work

1) General attention

As the concrete work for retrofitting construction is accomplished ordinarily by casting concrete little by little into different portions of the building, the construction plan shall be determined so as to obtain the required quality of concrete under the given conditions.

2) Selection of ready-mixed concrete plants

When the ready-mixed concrete is used, the concrete plant shall be selected so as to finish placing within the required time from mixing considering the time necessary to place concrete at the site.

3) Division of placing

Division of placing concrete shall be determined so as not to yield overwork for the planned working flow considering the conveyance method of concrete in the building, the time necessary to place and consolidate concrete at each placing part, the possible placing volume in a day, and the limit to the time from mixing to finishing of placing.

(2) Proportion

1) Specified compressive strength

Specified compressive strength shall be more than that of existing concrete.

2) Required slump

Required slump shall be less than 18 cm in principle, and as small as possible within the limit to placing.

3) Maximum value of water-cement ratio

The maximum value of the water-cement ratio shall be 65%.

4) Minimum value of unit cement quantity

The minimum value of the unit cement quantity shall be 300 kg/m^3.

(3) Preparation of Placing

1) Before placing, the chipped faces of existing concrete members shall be cleaned up by compressed air, absorption machine, cleaner and water or others.

2) Before placing, the faces to touch new concrete, such as those of sheathing boards and existing concrete faces, shall be adequately damped by water.

(4) Placing and Consolidation

1) Concrete is in principle placed from the opening of the slab of

upper story provided for placing.

2) When additional members or portions must be monolithically placed with existing portions of the upper story, two steps of placing or grouting shall be performed in principle leaving the space of 10 ∿ 20 cm at their upper parts.

3) Concrete shall be sufficiently consolidated by vibrators, and supplementarily by tamping and beating.

(5) Curing

Particularly careful wet curing of concrete of additional portions is necessary. Concrete shall be kept in wet condition over seven days when the expansive admixture is used.

(6) Forms

1) Forms shall be carefully constructed so as to keep the accuracy of the position and the size of additional members or portions. Care must be paid so that the leakage of mortar or others may not occur at the connecting parts between existing members and forms.

2) In the case of strengthening of columns by steel plates, attention must be paid so that the overhang of steel plates by lateral pressure of concrete may be prevented.

3.6 Mortal Work

(1) Scope

This section is applied to the work for mortar used for strengthening of columns.

(2) Proportion of Mortar

1) Compressive strength of mortar shall be more than the specified compressive strength of existing concrete.

2) Consistency shall be as stiff as possible in accordance with the placing portions and placing methods.

3) Standard proportion may be selected as shown in the following table in accordance with the mortar consistency examined by flow tests specified in JIS R 5201 (physical test methpds of mortal).

flow (mm)	cement : fine aggregate (weight ratio)
less than 180	1 : 3
more than or equal to 180, and less than 240	1 : 2.5
more than or equal to 240	1 : 2

(3) Placing or Spraying of Mortar

1) In the case of placing of mortar into forms or into strengthening steel plates, the mortar is placed from the upper part or pressed-in from the lower part so that the placed mortar may be even and dense.

2) The spray work of mortar follows JASS 15.

3) Before placing or spraying, the surfaces of existing concrete and sheathing boards shall be in sufficiently saturated condition with water.

(4) Curing

Mortar is cured in the same way as concrete.

3.7 Grout Work

(1) Scope

This section is applied to the grout work for connecting portions between existing concrete members and strengthening members such as those of upper parts of additional walls.

(2) Proportion

1) Compressive strength of grout mortar shall be more than the

specified compressive strength of concrete of strengthening members.

2) Consistency shall be determined according to the portions and methods of pouring.

(3) Production and Conveyance

1) Grout mixers by which even grout mortar is obtained shall be used.

2) Grout mortar shall be conveyed by the method which does not result in excessive separation of mortar.

(4) Pouring and Press-in

1) Grout mortar shall be placed by pouring or press-in.

2) Before the pouring or press-in, the surface of existing concrete and sheathing boards shall be in sufficiently saturated conditon with water.

3) Grout mortar shall be poured or pressed-in with adequate pressure without interruption.

4) Providing air vents, it shall be checked that grout mortar comes out through vents.

(5) Forms

1) Forms shall be constructed so as not to yield the leakage of grout mortar.

2) Forms must have the rigidity by which they may sufficiently resist the pressure associated with pouring or press-in of grout mortar. In addition, they shall adequately restrain the expansion pressure of grout mortar.

3) Forms shall be removed after grout mortar becomes adequately stiff and the restraint to the expansion pressure becomes unnecessary.

(6) Curing

Curing follows Section 3.5 (5), and particularly careful wet curing is necessary when the expansive admixture is used.

3.8 Plaster Work

(1) This section is applied to works for finishing plaster after the strengthening of structure.

(2) In the strengthening of columns by means of steel plates and filled mortar, adequate backing is needed in order to protect the separation of mortar when mortar cover finishing is provided on the steel plates.

3.9 Exterior and Interior Finish Work

(1) Waterproof finishing shall be provided for the exterior construction joint of new and old concrete.

(2) After retrofitting works, exterior and interior finish works are performed according to special specifications.

3.10 Inspection and Management of Quality

The inspection and management of the quality of materials and products are in principle in accordance with the standard specification "JASS" of AIJ. However, the lots for inspection and the number of pulling-out tests shall be determined so as to adequately indicate the quality of materials and products to be used.

REPAIR AND STRENGTHENING OF MASONRY

Lawrence F. Kahn[I]

SUMMARY

An epoxy cement and high-slump Portland cement grout are used to repair cracks and to fill voids in masonry structures. Unreinforced masonry walls and columns typically are strengthened with a surface application of shotcrete or plaster reinforced with vertical and horizontal bars and tied to the existing masonry with short dowels. Tying floors-to-walls and adjoining walls together with added connectors and exterior reinforcements increases a masonry building's seismic resistance.

STATE-OF-THE-ART

Practicing structural engineers and constructors innovate new repair and strengthening techniques for masonry structures with each job, for each building. The practitioners in North America and throughout the world, rather than academics and research engineers, have been the ones most involved in structural retrofit and its development. Those practioners write little concerning the procedures used for repair and strengthening and of their success, partly because they are so busy doing the engineering and partly because they do not want to bring attention to the distressed structure. Determination of the state-of-the-art, therefore, is based on few publications relative to actual retrofit construction and on a limited number of research studies.

In general, repair and strengthening techniques applied to brick masonry have been equally applicable to stone and concrete block construction and vice versa. In a series of shaking table tests on masonry houses, Gulkan et al. (12, 13) found that brick and concrete block structures responded similarly to seismic forces and that a surface bonding repair and strengthening method performed well on both brick and block.

Further, retrofit techniques for reinforced concrete structures are often the same as those for masonry. The special considerations for stone, block and clay products are the bond characteristics between lime/portland cement mortars and the masonry units, the typical multiple wythe construction of load bearing masonry, and the architectural use of the structural materials for load bearing and non-loading walls, infilled walls and veneers. The architectural considerations often place restrictions on the repair and strengthening procedures of masonry structures.

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Following sections of this paper will present methods which have been used for masonry repair and strengthening and which have been investigated in laboratory research efforts.

REPAIR

Cracked masonry walls typically are repaired by filling the cracks with a masonry grout or with an epoxy cement. The masonry grout combines portland cement, lime, and water into a high slump mixture. The grout rather than epoxy is used where cracks are wider than 1/8 inch and where economy is important. Cracks are sealed with a gel epoxy or with a dry-pack mortar. The masonry grout is then pumped under a low pressure into the cracks and voids in the structure. Considerable amounts of material flow into cavities between wythes of bricks and into existing voids between masonry units. The high slump is necessary to allow the grout to flow into the cracks and to sustain sufficient water in the grout for cement hydration after the bricks or blocks have absorbed much of the water from the grout.

Injection of low viscosity epoxy has been used successfully to bond cracks only 0.005 inch wide. But epoxy bonding is expensive for repair of masonry structures because the epoxy flows into voids and cavities between wythes as well as into the cracks. This flow requires large amounts of the costly epoxy (21).

Warner (27,28) has developed an expansive structural epoxy-ceramic foam which has high bond and compressive strength and which fills small cracks. Because of the foaming nature, less epoxy is required; and because of rapid set, leaking of the material through unsealed cracks is minimized.

In general, the repair of a damaged structure is designed to restore the structure's original strength. Benedetti and Costellani (4) showed that grouting existing stone masonry walls nearly doubled their lateral strength. In masonry such grout repair may result in strengthening because the grout fills voids and improves the bond and interaction between the masonry units. Sheppard and Tercelj (25) found similar strength improvements for pressure grouted stone masonry, whereas such repair of concrete block masonry only restored without increasing the original strength of test wall panels.

STRENGTHENING

Brick masonry industrial and multi-story buildings strengthened using simple techniques resist earthquake forces much better than similar, unstrengthened structures. Yaoxian and Xihui (29), Yuxian (31) and Guoliang (11) report that many brick structures were strengthened prior to the 1976 Tangshan, China Earthquake and that "...strengthened low-quality buildings behaved much better than those unstrengthened good-quality buildings" (11). Strengthening techniques include application of a reinforced shotcrete surface, reinforced plaster surface treatment, external reinforcing and post tensioned bars, reinforced concrete pilasters and bond beams, and improved floor and roof-to-wall connections.

It is difficult to determine to what degree an unreinforced masonry structure needs to be strengthened for improved earthquake resistance. A scientific understanding of the seismic response of masonry buildings is still being developed (18). Therefore, exactly how much steel and concrete reinforcement that is required to strengthen an existing building is not able to be quantified. Yet engineers have recognized some qualitative goals for seismic retrofit: to create redundancy and multiple load paths, to tie the building together so that it acts as a whole, to develop tensile capacity particularly for out-of-plane flexure in walls, and to develop ductility by keeping the masonry units together after the mortar joints have cracked. As the strengthening techniques are discussed in the following sections, these retrofit objectives should be remembered.

Reinforced Shotcrete

Application of a layer of shotcrete reinforced with vertical and horizontal bars is the recognized method for strengthening load bearing and non-structural masonry in North America. One example of its use was the seismic strengthening of the California State Capitol (16). The exterior brick load bearing walls were 84 ft. high and as much as 36 inches (8 wythes) thick. These were strengthened by first removing two interior wythes of brick, drilling holes into the remaining wall and anchoring steel dowels in them, placing reinforcing bars, and shooting a 12-inch thickness of concrete onto the interior surface (Figure 1). The new shotcrete wall was anchored to the existing concrete footing and to a new mat foundation. A second example of the use of shotcrete was the strengthening of the brick facade and the reinforced concrete elements of a six-story warehouse building in San Francisco (8). As Figure 2 shows the existing wythes of brick were anchored to the shotcrete by core drilling 8 in. diameter holes in the brick, embedding #4 bars, and filling the holes with dry-pack mortar. The masonry surfaces were sandblasted; after locating reinforcing steel, the shotcrete was applied in a 4-in. to 6-in. thickness.

In these two examples and others (3, 10, 14, 17, 20, 24), maintaining the architectural character of the structure was important. Application of the shotcrete to only the inside face of the masonry wall accomplished this requirement. The author has found no experimental research where this method of strengthening has been investigated.

Surface Treatments

Guoliang (11) reported that brick walls were strengthened after the Tangshan earthquake using reinforcing fabrics followed by a 1-in. to 2-in layer of mortar. When the Ninghe earthquake occurred, most of the strengthened buildings survived.

Schneider and Dickey (23) reported on tests of brick wall panels (3-wythes thick) which were strengthened with layers of wire mesh plus 1-in. thickness of plaster applied to each face. The mesh was either 0.10-in. or 0.06-in. diameter wire with 2-in. spacing vertically and horizontally. The shear strength of the strengthened specimens was more than twice that of the unstrengthened walls regardless of mesh size.

Jabarov, et al. (15) reported use of a similar strengthening technique and concluded that the 1-inch reinforced mortar layers increased the lateral strength of the masonry by a factor of 2.9.

Tso, et al. (26) used expanded metal sheets plus a thin layer of portland cement mortar to strengthen test specimens made using concrete block masonry. The first structure was reinforced with mesh on both sides; 1/4-in. diameter bolts passing through the wall held the mesh to the blocks and resulted in a "sandwich" construction. This first specimen showed a lateral resistance nearly three times greater than an unreinforced wall; reversed cycle deflections over increasing deflections resulted in constantly increasing energy dissipation. A second specimen strengrhened with mesh on only one side was about twice as strong as the unreinforced specimen. But the second structure was far less ductile than the first; the two-sided strengthening confined the ruptured masonry and prevented deterioration with increasing deflection cycles. A third specimen was tested and failed prior to strengthening, it was then strengthen with mesh and mortar identically to the first. Retesting showed a lateral strength about two-thirds that of the first, but it sustained reversed cycle deflections and showed ductility similar to the first.

Sheppard and Tercelj (25) strengthened concrete block walls on each side with a 1-in. layer of cement plaster reinforced with 1/4-in. diameter wires at 6-in. centers vertically and horizontally. The sandwich layers were joined together with 1/4-in diameter stirrups passing through the wall, spaced about 12-in. on center each way. The in-plane strength of the walls was twice that of unstrengthened structures.

A one-eighth inch thick layer of "Surewall", a proprietary material, was found to double the lateral strength of unreinforced concrete block walls. "Surewall" is a surface bonding cement made of portland cement, sand and alkali resistant glass fibers. A similar glass fiber reinforced plaster was used to repair and strengthen unreinforced concrete block walls of one-story structures which were subjected to shaking table tests (12). Walls were plastered with a 1/8-in. thick layer of the material after cracking had occurred at lateral accelerations of 0.28g. The repaired structures resisted further accelerations to 0.49g without failure. Gulkan et al. state, "the repair method restored the strength of both the in-plane and out-of-plane walls so that they were capable of resisting base motions significantly greater than those that caused the original damage" (12).

Altogether, surface treatments using lightly reinforced thin mortar layers have been shown to improve the lateral load capacity and the dutility of unreinforced masonry walls. Further, the bond between fairly low slump mortars and masonry seems sufficient to develop the strength of the steel reinforcement. Confining the existing masonry with surface treatments on both sides of the wall and with ties between each reinforced mortar face definitely was superior for increasing the strength and ductility.

External Reinforcing

Prestress and non-prestressed reinforcing bars mechanically attached to the exterior of existing masonry walls have greatly improved their lateral load resistance and ductility. Benedetti and Castellani (4) strengthened stone masonry structures with vertical tendons, horizontal tendons and with both (Figure 3). Structures were tested under monotonic, lateral loads.

About twice the lateral strength of unstrengthened walls was found for structures with vertical steel distributed along the length of the wall and for structures with vertical bars at the corners plus horizontal bars near the roof line and the base. The vertical tendons (.14-in.^{2} area) were prestressed to 3,300 lbs to give a nominal vertical prestress of 14 psi.

Other experiments on masonry walls and piers have shown that a superimposed bearing load on the structures increased their shear resistance; for bearing stresses less than 250 psi, the ductilities were increased also (18). Application of prestressed vertical bars seems to provide the same strength increases as these bearing stresses. External horizontal steel bands were used to repair and strengthen a single wythe brick wall in a shaking table test experiment (13). This retrofit permitted repeated shaking tests with effective peak accelerations near 0.5g; the bands maintained the structure's integrity.

The connection between exterior and/or interior walls framing together has been improved with external bars as illustrated in Figure 4. The improved connection forces the walls to act together in resisting the earthquake (22).

Reinforced Concrete Frame Elements

Masonry structural walls have been "tied together" using reinforced concrete elements built within and adjoining the existing structure. At Stanford University, Holmes (14) carved our areas in existing 24-in. thick, 3 story high unreinforced brick walls and then used shotcrete to place reinforced pilasters in the cavities. Churayan and Djahua (7) used a similar technique where they removed vertical sections in brick walls, placed reinforcement and cast in concrete made with crushed brick as aggregate; the connection of these pilasters to the brick was achieved by maintaining an irregular, saw-tooth surface in the remaining brick wall. Lee, et al. (16) cast a reinforced concrete bond beam at the top of the brick walls for the California Capitol, which was also strengthened with a shotcrete surface (Figure 1). And Jin Guoliang (11) stated that one Chinese strengthening method for brick walls was casting "reinforced concrete columns which are securely connected to the walls."

Reinforced bond beams are normally used in new construction to join the exterior load bearing walls and to provide a good connection to the wall-to-roof diaphram. Casting such a member to provide structural continuity in existing walls is a standard procedure (10, 22).

Improved Connections

Secure connections between floors or roof diaphrams and load bearing masonry walls greatly improve the seismic resistance of masonry buildings. One such connection uses a joist anchor as shown in Figure 5. Joist anchors were used to strengthen brick masonry buildings in Los Angeles, California area prior to the February 9, 1971 San Fernando earthquake. Abel (1) points out that walls so strengthened generally remained standing while similar unstrengthened wall collapsed during the 1971 event. Briasco (5) found that no walls with joist anchors separated from the floor or roof during the San Fernando earthquake.

Guoliang (11) writes that several months before the Tangshan earthquake the connections between infilled masonry walls, columns and roof trusses were strengthened by bolts. As a result, none of these structures collapsed, and few infills were broken.

Improving connection details to force walls to move with the floor and roof diaphrams seems a low cost method which greatly increases a buildings' seismic resistance. Roofs have collapsed because the load bearing walls on the opposite sides of a building moved in opposite directions during the earthquake (5). Securely connecting each wall to the roof forces the walls to deflect in similar directions, thus the walls remain under the roof structure and collapse is prevented.

CURRENT RESEARCH EFFORTS

Ewing and others (9) are conducting a significant experimental effort to determine the response of masonry structures. Tests will examine diaphrams, out-of-plane and in-plant response of walls and anchorages. After initial tests, the full-scale specimens will be repaired and strengthened, then retested. Dr. Russell Brown at Clemson University is experimentally investigating anchorages in masonry walls, including the use of joist anchors for seismic retrofit. The author is studying the bond characteristics between shotcrete and brick walls (Figure 6). Tests will determine the degree of composite response between the shotcrete and masonry and the extent to which the reinforced shotcrete strengthens brick elements.

CONCLUSION

In general the various repair and strengthening techniques attempt to bond the masonry units together better and to provide steel reinforcement for tensile resistance and for ductility. Pressure grouting masonry where the mortar has cracked and deteriorated results in strengthening the wall structure. Surface treatment seems to double the in-plane strength of brick and block walls; typically for such surfaces, small quantities of thin steel wire fabric are bonded to the surface of walls with less than a one-inch coat of cement plaster. Where unreinforced masonry walls have been tied together with extra joist anchor connections, external rods, bond beams, pilasters, or some surface treatment, the walls and buildings have resisted seismic forces. Similar structures not so strengthened have collapsed.

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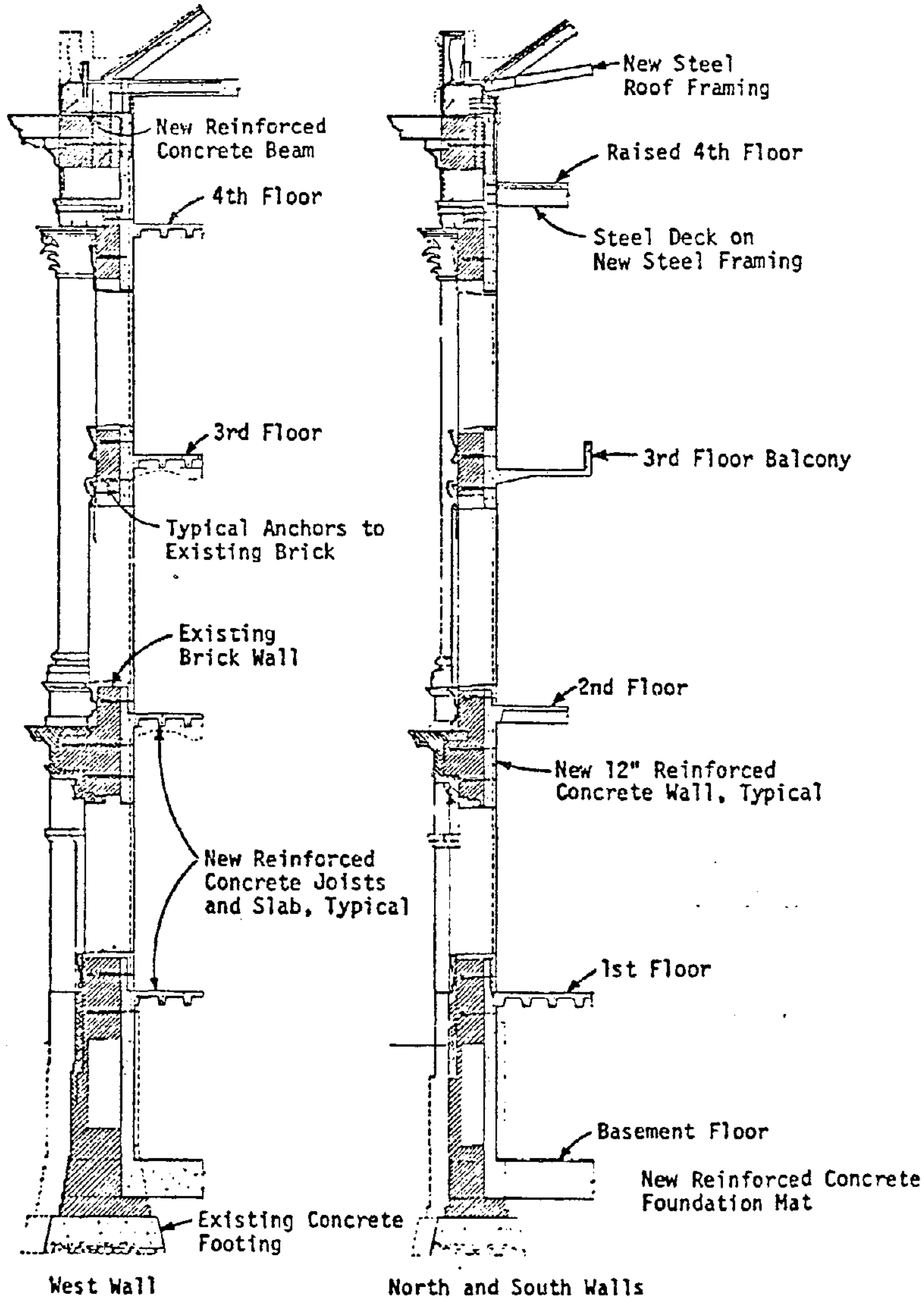

Figure 1. Shotcrete strengthening of California State Capitol (from Ref. 16)

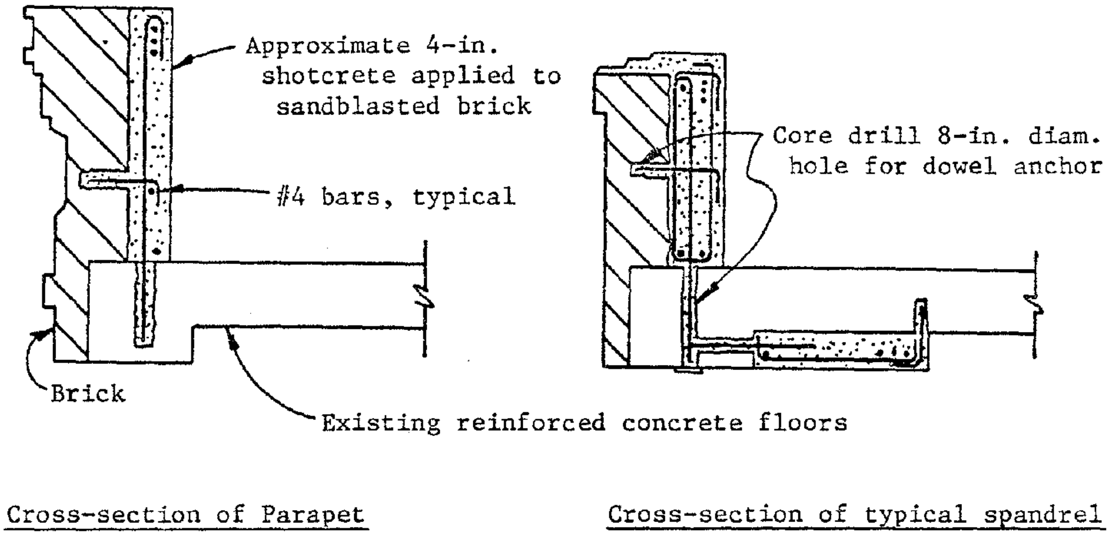

Figure 2. Reinforced shotcrete used to strengthen brick facade and the reinforced concrete structure (from Ref. 8)

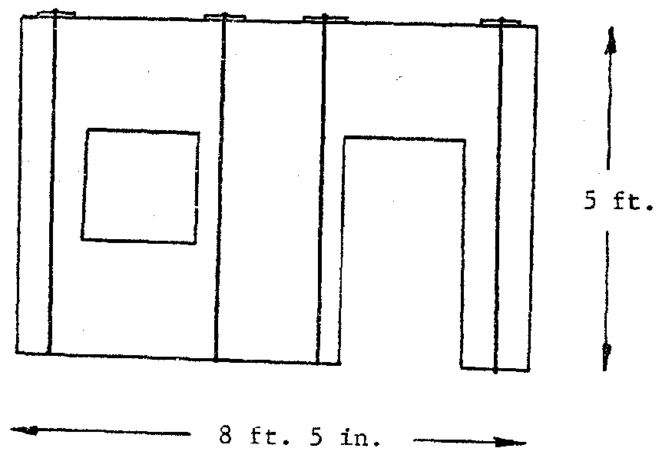

Figure 3. External vertical tendons used to prestress stone masonry wall specimens (from Ref. 4)

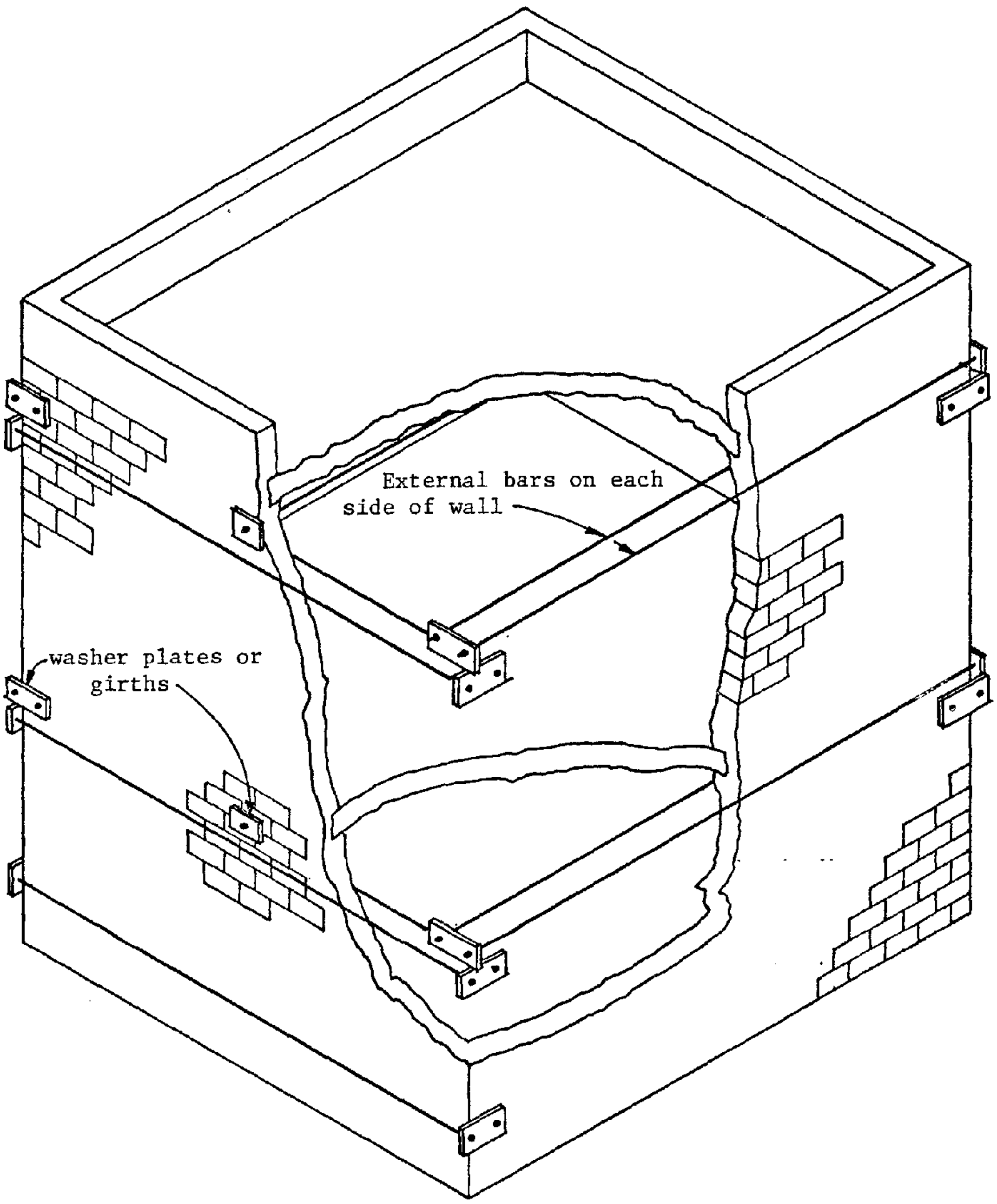

Figure 4. External reinforcement and/or prestressing tendons used to connect transverse walls and walls-to-floors.

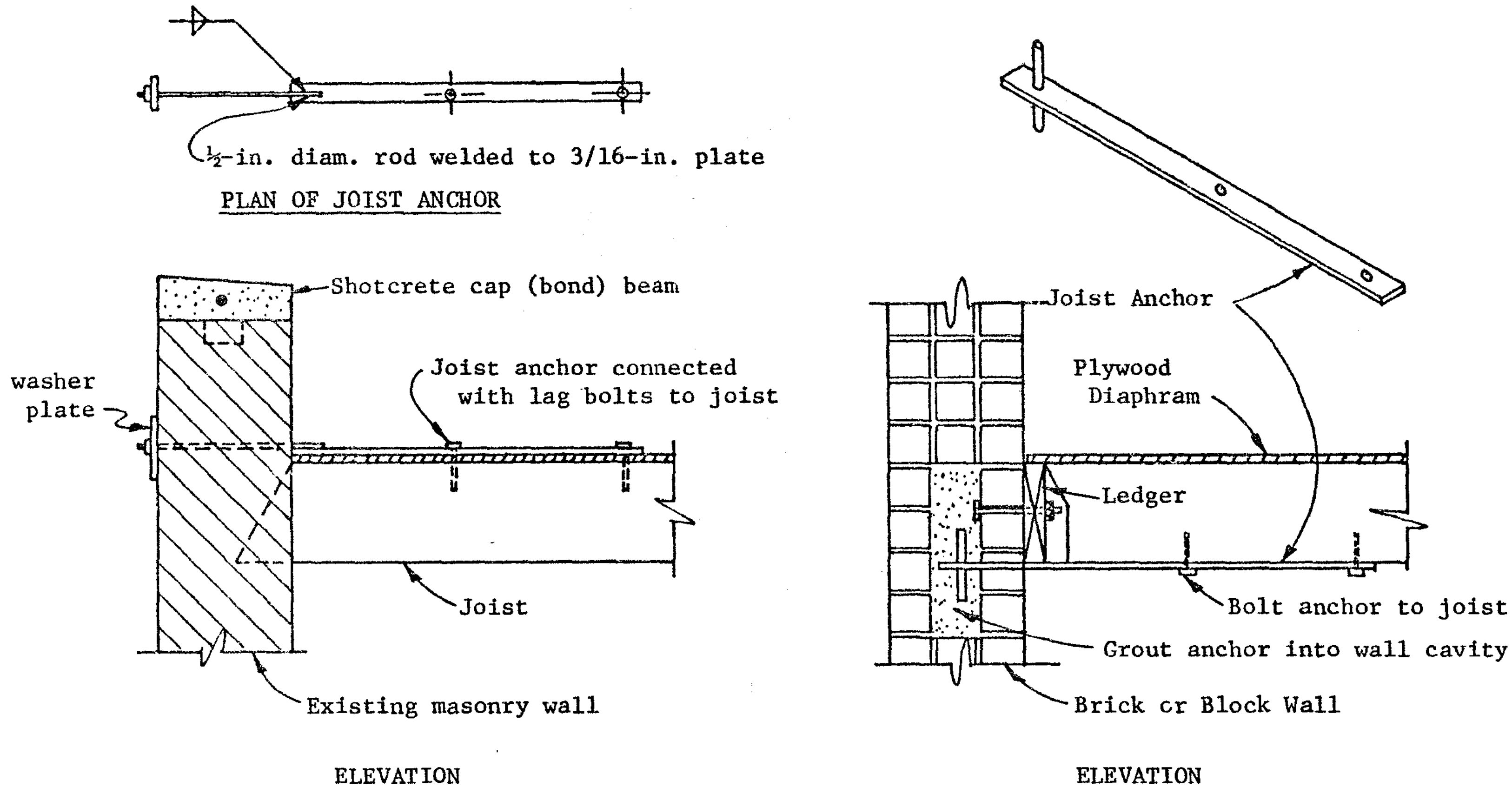

Figure 5. Joist anchors used to connect existing roof and floor diaphram to masonry wall (from Ref. 1).

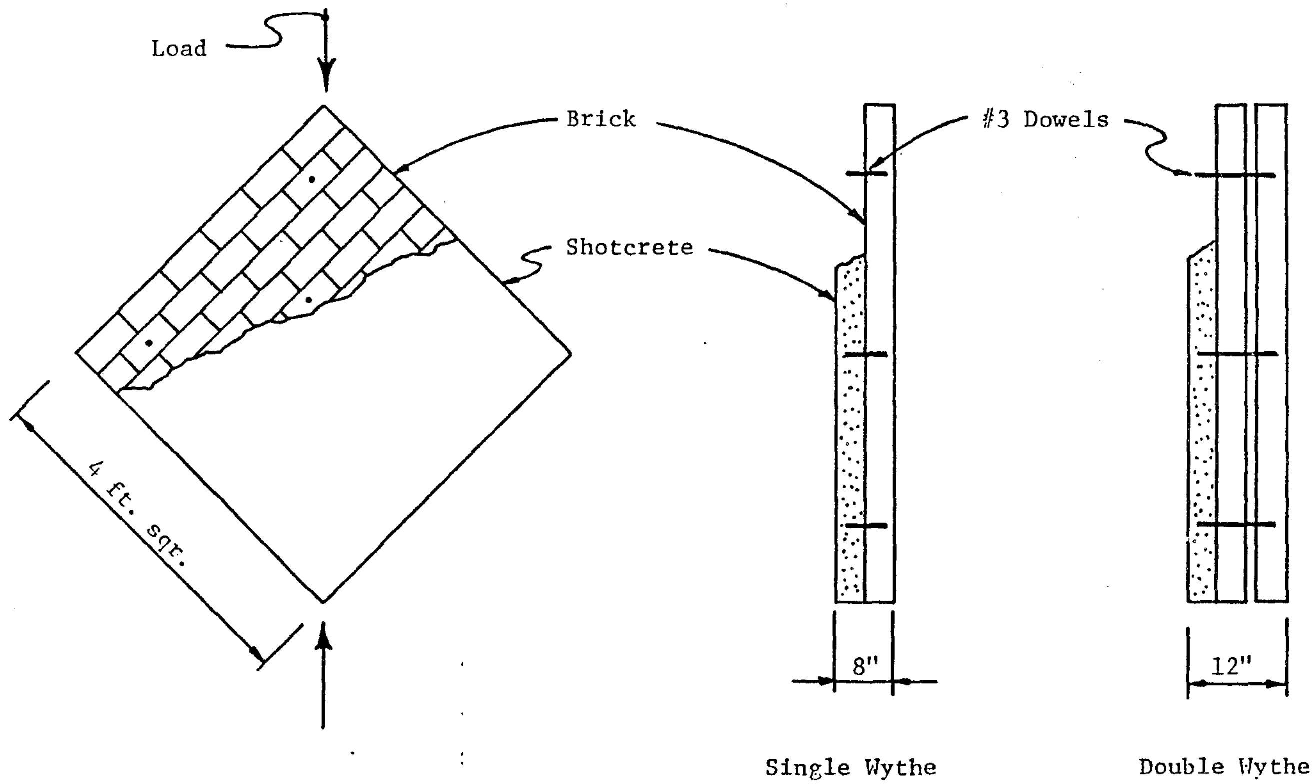

Figure 6. Diagonal tension (shear) test of composite brick-shotcrete specimens.

STRENGHTENING EFFECT OF ECCENTRIC STEEL BRACES TO EXISTING REINFORCED CONCRETE FRAMES

Shigeya Kawamata [I] and Masaaki Ohnuma [II]

SUMMARY

One of the main buildings of the Tohoku Institute of Technology in Sendai, which was damaged by the '78 Miyagi-ken-oki earthquake, was restored. The eight storied reinforced concrete frame construction was strengthened in the longitudinal direction by means of steel cross braces which were installed with eccentricity in both façades from outside of the building.

In this paper, the scheme of the bracing is described and the results of experimental works on the behavior of the eccentric cross braces, strength of brace-to-frame connection and on spandrel weakening device are presented. Also, the aseismic effect of the bracing system to the building is evaluated.

INTRODUCTION

Two of the main buildings of the Tohoku Institute of Technology in Sendai, Japan, were seriously damaged by the Miyagi-ken-oki earthquake of June 12, 1978 [1]. One of them, Building No.3, four storied reinforced concrete frame construction, was judged irrecoverable, demolished and being reconstructed. The other, Building No.5, which was eight storied R.C. frame construction (Figs.1 and 4) was repaired, strengthened and resumed its service in ten months after the earthquake.

The damage to both the buildings is characterized by the same mode of failure, i.e. shear and bending-shearing failure of columns in the north side frame under the action of horizontal force in the longitudinal direction (Figs.2 and 3). One cause of the destruction is supposed to be the deficiency of ultimate strength of the frames, none of or very few shear walls existing in this direction. Another and more important factor is the influence of in-fill spandrel wall. The walls, being cast-in-place only in the north frame as shown in Fig.1, made the stiffness of the frame about four times greater than those of the other two frames and gave rise to concentration of shearing force , thus resulting the brittle shear failure of the columns.

The latter factor had been ignored in the design of buildings until the dangerous effect of in-fill spandrel walls was drastically recognized by the destruction of Hachinohe Technical College in 1968 Tokachi-oki earthquake [2],[3] (the Buildings No.3 and No.5 were built in 1966 and 1968 respectively).

For the Building No.5, not only the failed and cracked columns, beams, walls and slabs were repaired, but the strengthening of the original frames together with the weakening of the spandrel wall was done to improve the resistance against big earthquakes expected in the future.

Strengthening of the building in the transverse direction was made by installing additional R.C. shear walls as shown in Fig.4. As the strengthening in the longitudinal direction, steel cross braces were attached to both faces of the building from outside. Rigid connection between

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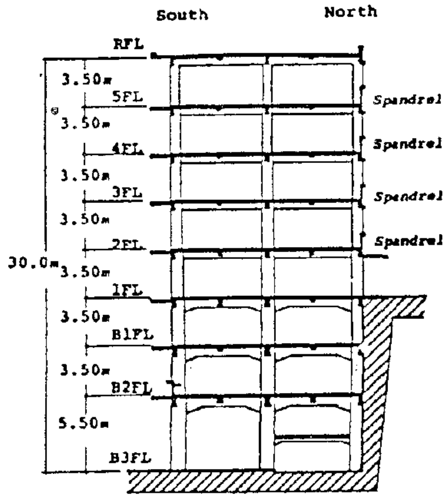


Fig.1 Section of Building No.5

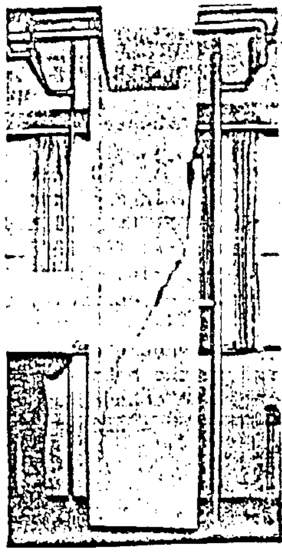


Fig.2 Typical cracks of columns

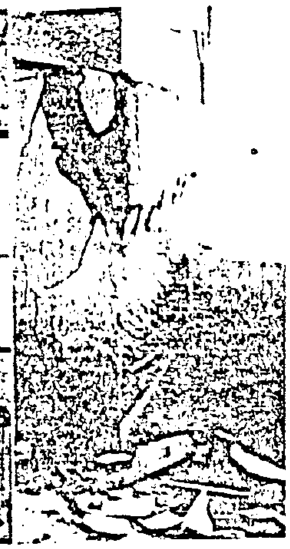


Fig.3 The most heavy damage to column

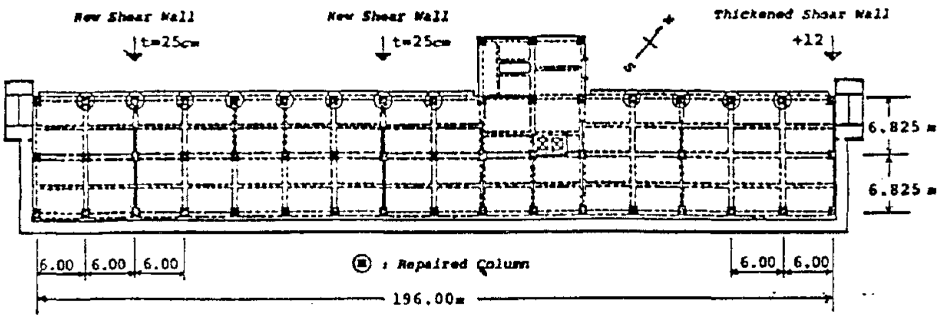


Fig. 4 Repaired columns and added shear walls (plan of 3F)

steel braces and the existing R.C. frames was secured by a kind of post-tentioning technique. Further, the ends of the in-fill spandrel wall were continuously perforated by core boring so that the columns may not encounter any more shear failure.

The authors were in charge of the restoration works as the members of a committee set in the Institute, and as such strengthening of existing R.C. frames by steel braces was a new technique which had not been experienced, a series of experiments was needed to verify the aseismic effect of the system.

In this paper, the design of the bracing system is described and results of tests on the behavior of braces, brace-to-frame connection and the weakened spandrel beams are presented. Basing on the experimental data, the strengthening effect of the bracing system is evaluated.

1. DESIGN OF STRENGTHENING BRACE SYSTEM

Basic Concept Though the installation of shear walls is the most common practice as aseismic strenghening for R.C. frame construction, a

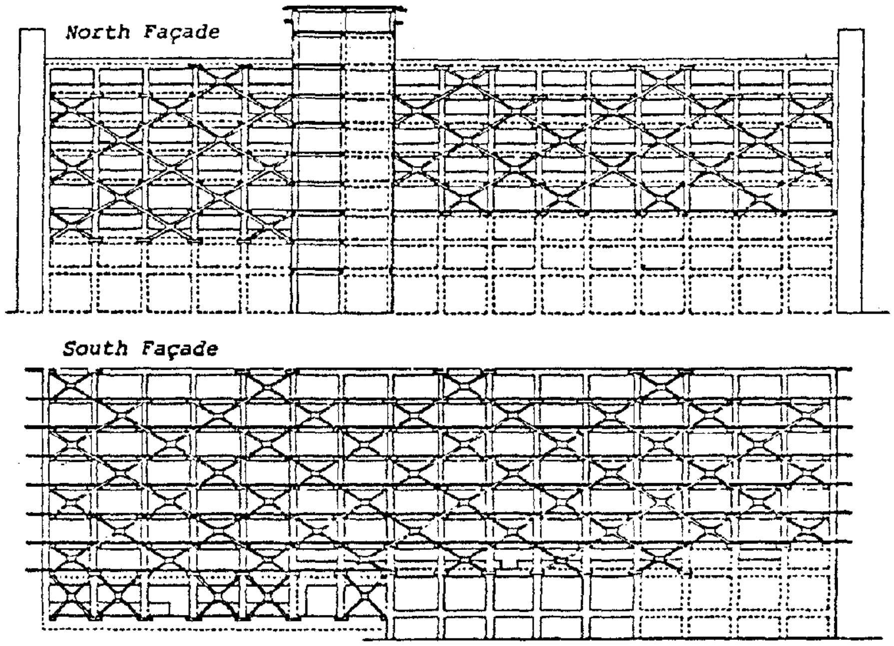


Fig. 5 Arrangement of cross braces

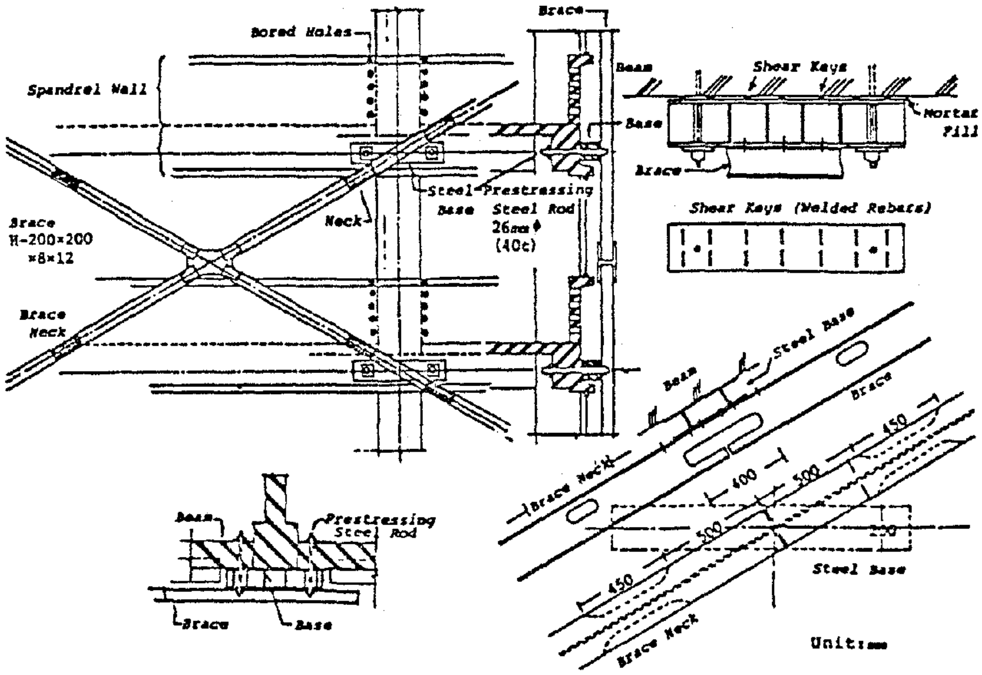


Fig. 6 Attachment of braces to existing R.C. frame

system of steel braces was adopted in the restoration work. The advantage of the system can be summarized in three points: a) natural lighting through windows is not intercepted, b) installation is approached from outside, thus facilitating the work and not making any obstacle in the interior of the building, and c) uiform distribution of braces is possible so that no concentration of shearing force occurs.

Brace to frame connection As shown in Fig.6, brace members were fastened by friction bolts to steel bases which were set against the R.C. beam face and, after filling the gap with cement mortar, post-tentioned by prestressing steel rods inserted through bored holes.

Brace Members H-section (H-200×200×8×12, mm) of weathering steel (JIS SMA41A, σ_Y=35kg/mm^2 and σ_B=49kg/mm^2) was used with coating of a rust stabilizing agent because the braces were to face the open air.

In each brace member, the outer flange was cut at the nodal point, as shown in Fig.6, in order to derive fully eccentric property. Further, as shown in Fig.7, in B-3 braces used for 4th and 5th stories, the neck section having narrowed inner flange and perforated web was formed. B-2 braces for 1st to 3rd stories had narrowed inner flange and B-1 braces for B3 to B1 stories had not the neck section. These necks were for accelerating the yielding under axial forces.

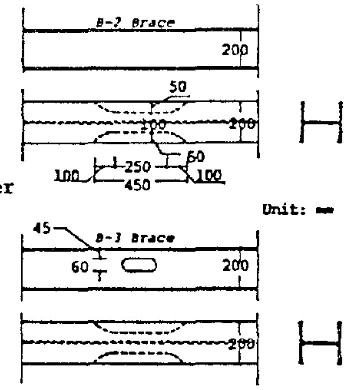


Fig. 7 Brace necks

Amount of Steel Used and Period of Execution

The total amount of steel sections and plates used for the bracing system was about 50 tons. Four months were needed for the fabrication and the installation.

2. HYSTERETIC CHARACTERISTIC OF CROSS BRACES

One third scale models were used in order to investigate the hysteretic behavior of the units of cross braces of the three different types. Brace section of 75mm×75mm was built up from 4.2mm thick sheet (σ_Y=32kg/mm^2 and σ_B=48kg/mm^2) for the flange and 3.0mm thick sheet (σ_Y=25kg/mm^2 and σ_B=34kg/mm^2) for the web.

As shown in Fig.8, alternate horizontal load was applied through a hinged rigid frame. The specimens were brought to their ultimate state by 9 to 10 cycles of loading with monotonically incrasing amplitude. Fig.9 shows the load- horizontal displacement curves for the point E in Fig.8.

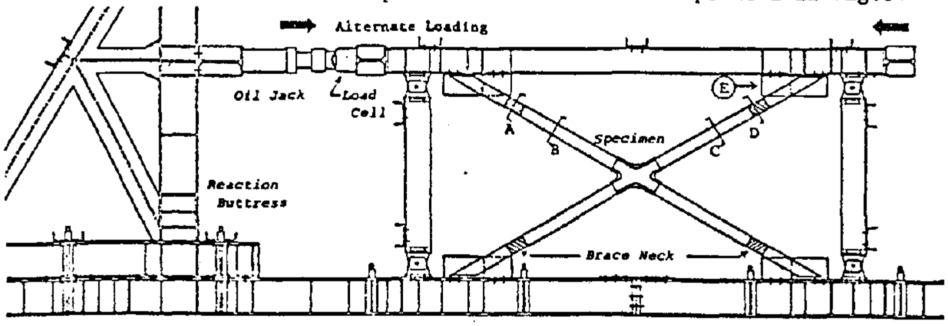


Fig. 8 Test setup for 1/3 scale models of cross braces

The behavior of the braces are characterized by the eccentric nature in the action of axial forces. As shown in Fig.10, the eccentricity is especially dominant in the neck sections (sections A and D in Figs.8 and 10) In the consequence of the eccentricity, the elastic stiffness to the lateral force was reduced to about 40% of the case in which the same members were subjected to concentric action of the axial force.

Yielding both in tention and compression initiated from the inner flange in the neck section because of the eccentricity, and gradually propapagated outward up to the outer extreme of the web, the outer flange remaining almost stress free or well in elastic range even in the ultimate state.

The ultimate bearing capacity was determined by the buckling of the inner flange and web in the neck section. However, the reduction of the load level was very small as can be seen in Fig.9. This is due to the localized nature of the buckling. The estimation of ultimate loads *Pult*, which were obtained under the assumption that inner flange and web in the neck were in the stress level of tensile strength/yield stress in tension members and of yield stress in compression members, outer flange being assumed to be stress free, is in good coincidence with the experimental values.

The property of the braces is summarized in Table 1.

Y.P.: Yielding of Inner Flange in Neck Section
L.B.: Initiation of Local Buckling in Neck Section

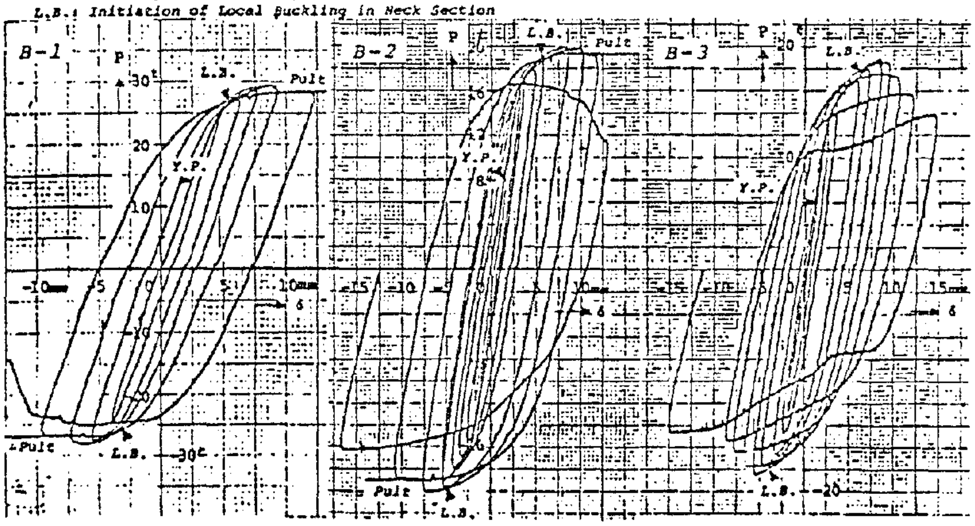


Fig. 9 Load-displacement curves (Point E in Fig.8)

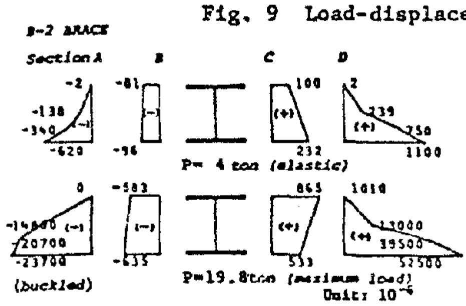


Fig.10 Measured axial strain

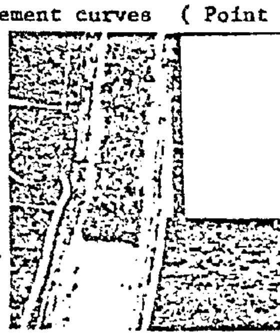


Fig.11 Buckling of Neck (B-2)

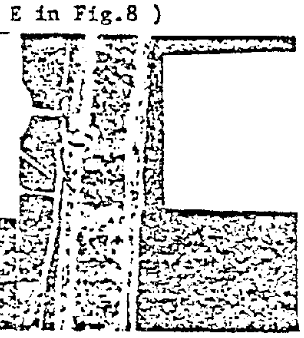


Fig.12 Fracture of Neck (B-2)

Table 1 Summary of Experimental Results of Cross Braces

Specimen	Initial Stiffness *1 χ	Initial Yielding *2 Load P_y, ton	Displacement δ_y, mm	$R=\delta_y/H$ *3 10^{-3} rad	Max. Load ton	Ultimate State Load P_{ult}, ton	Displacement δ_{ult}, mm	$R=\delta_{ult}/H$ *3 10^{-3} rad
B-1	37	14. (117)	2.1 (6.3)	1.83	29.3	28.4 (255)	9.8 (31.2)	8.4
B-2	40	8. (71)	1.3 (3.9)	1.09	19.8	19.0 (181)	10.8 (32.4)	9.3
B-3	40	6. (54)	1.0 (3.0)	0.86	18.7	15.6 (156)	10.4 (29.4)	8.9

(----) : *Figure converted for the corresponding prototype braces*
*1 : *Lateral stiffness divided by theoretical stiffness for the case of concentric cross braces*
*2 : *The first yielding of inner flange in the neck section*
*3 : *H is the height of story*

3. STRENGTH OF BRACE-TO-FRAME CONNECTION

The strength of brace-to-frame connection is governed by the shearing capacity of the junction of steel base and R.C. beam face under the existence of the transverse prestress. In order to prove the reliability of the joint, a series of slip tests was carried out. Scale of the specimens was 1/2. Fig.13 shows the test setup. The tests were performed under two kinds of loading: the loading in the direction parallel to the axis of base and the one perpendicular to the axis.

Fig.14 shows a load-slip displacement curve for a specimen loaded longitudinally. The junctions behaved very well both in strength and in ductility in virtue of the prestress. In the ultimate staue, the coefficient of friction with respect to the induced prestress reached to 1.6-2.0 in the cases of longitudinal loading and to 1.1 in the cases of transverse loading. In the context of the prototype, the maximum hrizontal force which can be transmitted from frame to brace is estimated to be 120*ton*

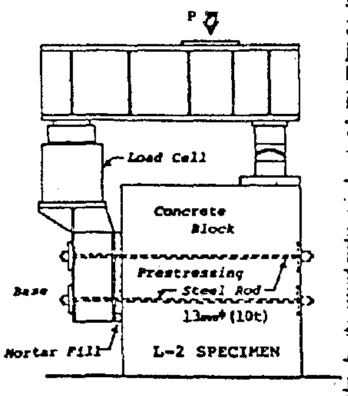


Fig.13 Slip test of steel base

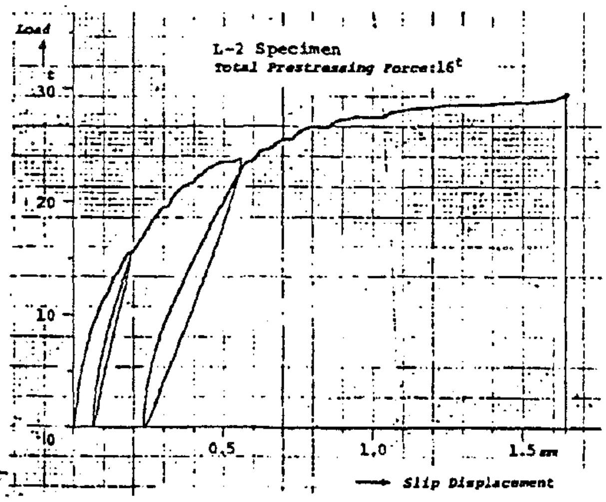


Fig.14 Load-slip displacement curve

in each of the general nodes (2 prestressing rods) and 240*ton* in the end anchorages (4 prestressing rods), which mean enough resistance in view of the ultimate capacity of braces indicated in Table 1.

4. BEHAVIOR OF WEAKENED SPANDREL WALL

In order to observe the performance of the spandrel walls whose ends were weakenend by bored holes, a pure beam and beams having spandrels with and without the holes were subjected to alternate bending moment and shear in the form of simple beam test. Scale of the specimens was 1/3 (Fig.17).

Fig.15 shows load-deflection curve of the beam with weakened spandrel(A-2) compared with the backbone curves of the other two.

For the bending moment in which the spandrel wall was in compression, the resistance of the weakened spandrel beam was reduced to about 1/3 of the unweakened one as the result of crushing of concrete remaining between the holes.*1

In the context of the framed configuration, it can be proved that the level of horizontal force corresponding to the spandrel crushing is well under the level of the one of the shear failure of the column, thus enabling us to avoid the latter.

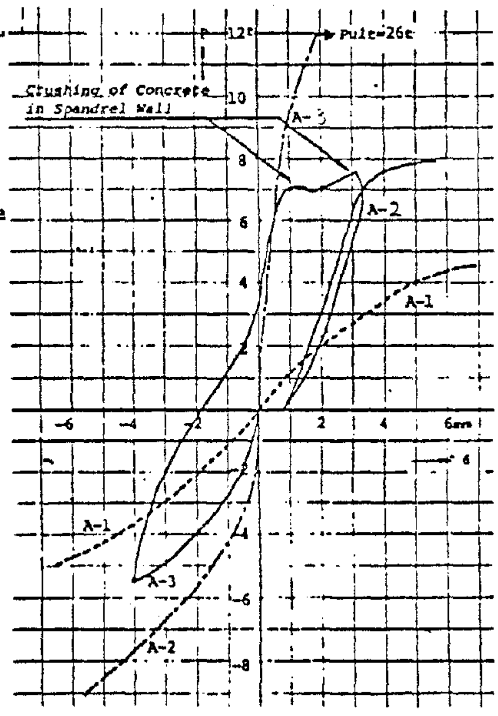


Fig. 15 load-deflection curve

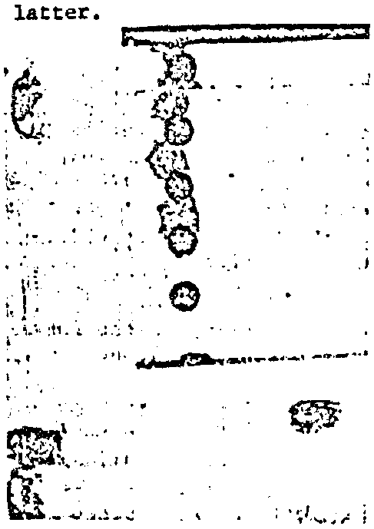
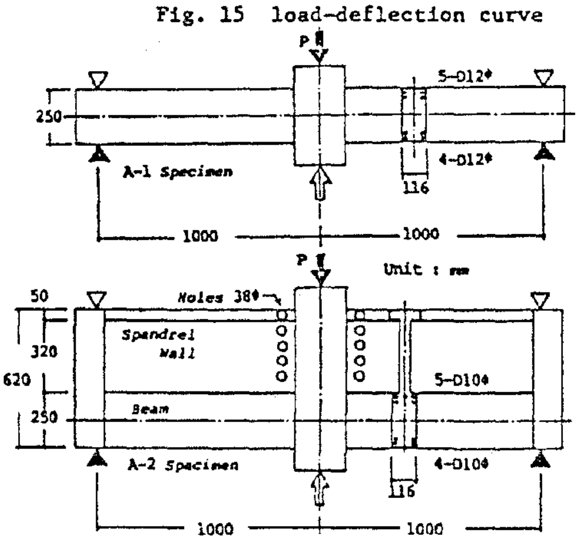


Fig. 16 Crushing of concrete around holes



A-3 Specimen is the same as A-2 but without holes

Fig. 17 Test specimens

*1 The wall concrete between holes was grooved with only outmost layer of 1cm thick (3cm in prototype) being left.

5. EVALUATION OF STRENGTHENING EFFECT OF BRACES : CONCLUSION

Table 2 shows the variation of natural period of vibration of Building No.5 before and after the earthquake obtained from Y. Abe's microtremor measurements [4]. The stiffness of the building in the longitudinal direction, which was once reduced to 40% of the original value, was almost totally recovered. Shortening of the period from 0.49*sec* in Feb. 1979 to 0.35*sec* in Apr. 1979 means that the R.C. frame stiffness became twice by installing the steel bracing system.

As the ultimate deformability of braces of about 1/100 of the story height is probably similar to the one of the R.C. frames, the summation of strength of the frames and braces seems to be reasonable. As shown in Table 3, the ultimate shearing force coefficients also became twice by the bracing.

This level of earthquake responce will hardly be reached in view of such large capacity of energy dissipation of braces as exhibited in the experiments.

Table 2 Natural Period of Building No.5

Date of Observation	Direction T	Direction L
Apr., 1975	0.39	0.34
Feb.22, 1978: after an earthquake of scale IV	0.39	0.44
June 28,1978: after Miyagi-ken Oki earthquake	0.43	0.53
Feb.13, 1979: after restoration of columns	0.36	0.49
Apr. , 1979: after installation of braces	0.36	0.35

unit: sec T : Transverse, L : Longitudinal

Table 3. Estimation of Ultimate Shearing Force : Longitudinal Direction

Story	Weight		Before Earthquake		After Strengthening			
			Shear.F.	Coeff.	Frames	Braces	F + B	Coeff.
	W, t	ΣW, t *1	Q_c, t *2	k_c *3	Q_F, t *5	Q_B, t	(Q_F+Q_B)	k *4
5	1,593	1,979	1,550	0.78	1,648	1,092	2,740	1.39
4	1,417	3,396	1,710	0.50	1,755	2,496	4,251	1.25
3	1,440	4,836	1,899	0.39	2,131	2,534	4,665	0.96
2	1,447	6,283	2,414	0.38	2,496	2,896	4,012	0.86
1	1,820	8,103	2,962	0.37	2,962	2,534	5,498	0.68

*1 including weight of penthouses
*2 shearing force to whole building corresponding to shear failure of north frame columns, including resistance of shear walls
*3 $k_c = Q_c / \Sigma W$, *4 $k = (Q_F + Q_B)/\Sigma W$
*4 shear failure of columns assumed not to occur

ACKNOWLEDGEMENT

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SEISMIC EVALUATION AND STRENGTHENING OF EXISTING MULTISTORY RESIDENTIAL BUILDINGS

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U.S. Department of Housing and Urban Development

ABSTRACT

In 1976 the U.S. Department of Housing and Urban Development (HUD) issued a contract to develop a methodology for seismic evaluation of existing multistory buildings. This methodology was based on a procedure previously developed by the National Bureau of Standards (NBS), and reported in NBS Building Science Series, BSS61, January 1975. HUD requires an evaluation of earthquake hazard and seismic resistance of all buildings located in Seismic Zone 3 (Uniform Building Code). This paper briefly describes the application of the Methodology for evaluating the seismic resistance of three typical multistory buildings in California (seven to twenty-seven stories high), and also the strengthening techniques used to rehabilitate the structures.

INTRODUCTION

In January 1975 the National Bureau of Standards (NBS) published a study "Natural Hazards Evaluation of Existing Buildings (BSS-61)" by Charles G. Culver and H.S. Lew of NBS, Gary C. Hart of J.H. Wiggins Company, and Clarkson W. Pinkham of S.B. Barnes and Associates [1]. This study presented a methodology for evaluation of damage to both structural and nonstructural building components resulting from extreme natural environments such as earthquakes, hurricanes and tornadoes. Three sets of procedures were presented: (1) Qualitative determination of damage level on the basis of data collected in field survey of a building; (2) Determination of damage level as a function of behavior of critical elements based on a structural analysis of a building; and (3) Determination of damage level based on a computer analysis of the entire structure.

Based on the second procedure presented by NBS, the U.S. Department of Housing and Urban Development (HUD) in June 1976 awarded a contract to S.B. Barnes and Associates to develop a methodology for seismic evaluation of existing buildings. This resulted in a three-volume manual, "A Methodology for Seismic Evaluation of Existing Multistory Residential Buildings" by Clarkson W. Pinkham of S.B. Barnes and Associates and Gary C. Hart of J.H. Wiggins Company, which was published by HUD in November 1978 [2]. Contained in the manual are methods of structural analysis, strengthening and repair of existing structures, cost analysis of remedial methods, and examples which illustrate both a simplified and more complex (computer) evaluation of stress distribution of different types of multistory buildings.

The HUD Methodology was limited to evaluation of seismic resistance of residential type buildings, but it also expanded the BSS-61 procedures by adding strengthening and repair as well as cost analysis to the scope. In addition, a unique computer analysis program was developed. This HUD Manual and its application were previously presented by Mr. Fuller at the Workshop on Earthquake-Resistant Repair and Retrofit of Buildings during the UJNR Twelfth Joint Meeting in Los Angeles on May 16 - 17, 1980 [3]. Two examples of buildings evaluated using the methodology [2] were discussed by Mr. Fuller.

At present, HUD requires an evaluation of earthquake hazard and seismic resistance of structural components for all buildings located in Seismic Zone 3 (Uniform Building Code, 1973 Edition [4]) in accordance with HUD Handbook 4940.4, Minimum Design Standards for Rehabilitation for Residential Properties, February 1978 [5]. See Figure No. 1 for Seismic Zone 3 locations in the United States.

EVALUATION AND STRENGTHENING

Several major cities in the U.S. other than Los Angeles and San Francisco, California are located in Zone 3, such as Boston, MA; Buffalo, NY; Charleston, SC; Memphis, TN; Salt Lake City, UT; Reno, NV; and Seattle, WA. Therefore, HUD has required an analysis of seismic resistance of several existing buildings intended for conversion to residential use under HUD programs. Most of the buildings have been in California; three of these are described herein.

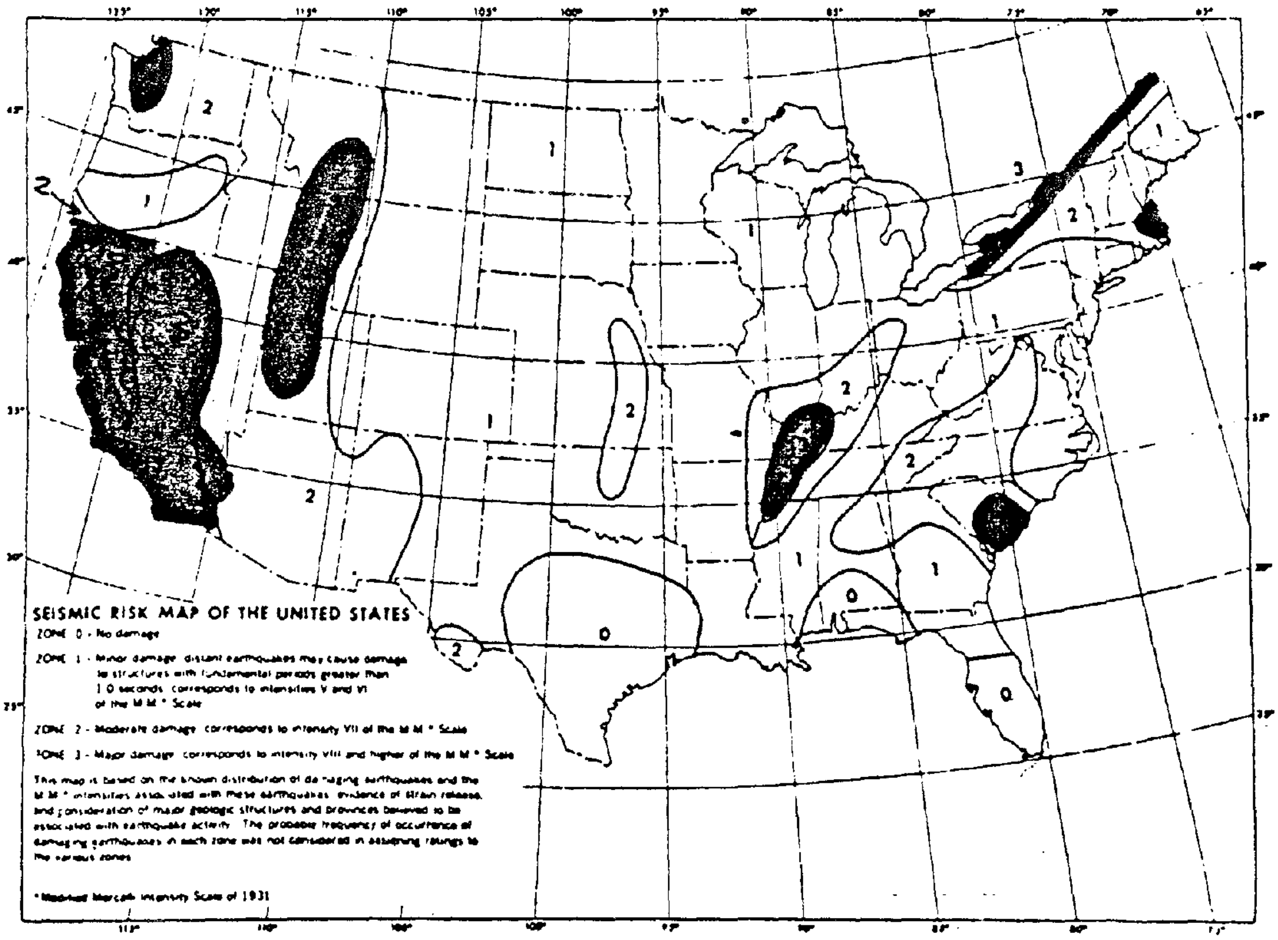

Figure No. 1

SEISMIC RISK MAP

*Note

Puerto Rico and Virgin Islands ... Zone 3

<u>Oakland Hotel, Oakland, California</u>

This was formerly a 400-room hotel which was to be converted to housing for the elderly. It is an eight-story building constructed in 1912 of steel and reinforced concrete (see Figure No. 2).

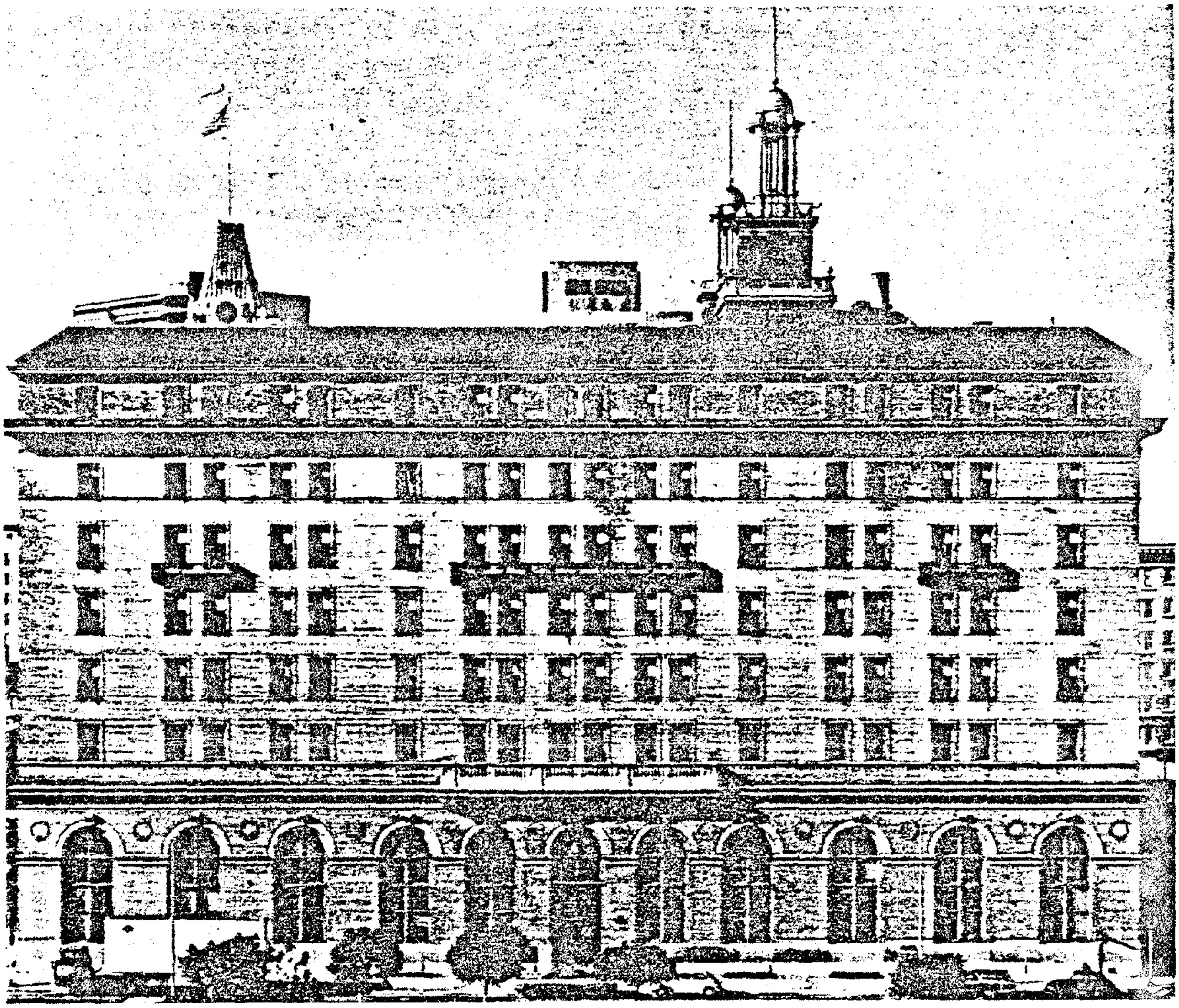

Figure No. 2 *The Oakland Hotel, Oakland, California*

It was determined by the Structural Engineer that critical elements in this case were first-floor columns. The Critical Stress Ration (which is the indicator for evaluating the seismic resisting capability of the structure) was 3.6 (28% with respect to UBC '73). The optimum Critical Stress Ratio is 1.0; therefore, the building would be overstressed by 3.6 times its capacity if subjected to a maximum credible earthquake. To remedy the situation reinforced concrete shear walls, 8" to 12" (10.32 - 30.48 cm) thick, were installed on all floors; and a footing supporting three columns was enlarged to prevent overturning.

After remodeling, 315 units of subsidized elderly housing were created. The Cafe, Club Room, Dining Room and Ball Room were retained for use by the residents. Both evaluation and construction have been completed, and the building is fully occupied. The cost of rehabilitation was $14,400,000 ($1,280,000 for the structural work).

Young Women's Christian Association (YWCA) Building, San Francisco

This seven-story reinforced concrete building was constructed in 1931 as a YWCA dormitory. Evaluation of this building using the Methodology revealed that critical elements were first-floor columns and corridor walls. The Critical Stress Ratio was 3.1 (32% with respect to UBC '73). To increase resistance to earthquakes, the exterior walls were strengthened by pneumatically applied concrete, 4" to 8" (10.16 - 20.32 cm) thick. This strengthening brought the building up to 100% compliance with UBC '73 (Critical Stress Ratio equal to 1.0).

During removal of corridor walls on all floors, large reinforced concrete trusses were uncovered, which indicates that the original designer was aware of the necessity for the building to be capable of resisting lateral forces.

The YWCA building is now fully occupied as housing for the elderly with 98 units. The total remodeling cost was $4,600,000 ($1,200,000 for structural work, including a special soil investigation).

<u>William Taylor Hotel, San Francisco, California</u>

The third building evaluated by the HUD Methodology is the old William Taylor Hotel located at 100 McAllister Street. This 27-story steel frame building with in-fill brick walls was constructed in 1929. For many years the building was used for government offices, but it is now being rehabilitated into a student dormitory for Hastings College of Law.

By making a conservative assumption - disregarding the resistance provided by brick walls but taking into account their weight, the Structural Engineer established the Critical Stress Ratio in beam-column connections as 4.5 (22% with respect to UBC '73). Beam-column connections located at the 14th floor and above were identified as the critical elements. It was recommended that framing at the 14th, 21st and 25th floor vertical offsets be reinforced by installing additional steel floors and by providing collector elements.

Evaluation of the building has been completed and demolition work has just started, but the total construction price has not been determined.

CONCLUSIONS

The HUD Methodology is based on a procedure which determines actual damage level as a function of behavior of critical elements. Then recommendations and cost estimates are made to bring the structures up to 25%, 50%, 75% and 100% compliance with the Code. The procedure outlined in the HUD methodology provides the evaluator with the information necessary to arrive at appropriate decision. The final decision as to the extent of rehabilitation must weigh the risk of loss of life, damage to property and importance of the project against cost of rehabilitation. Structural engineers experienced in the design and analysis of structures capable of resisting seismic forces can use their own approach. However, the primary purpose of the HUD methodology is to provide a tool to structural engineers not necessarily familiar with aseismic analysis. The fact that some of the most experienced engineers applied the basic concepts of the HUD methodology underscores its value and usefulness.

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RETROFITTING ON MEDIUM - RISE REINFORCED CONCRETE HOUSING STRUCTURES

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ABSTRACT

In this paper, aseismic safety of two existing medium-rise R.C. housing structures which have the soft 1st story is discussed. This type of structures generally has a possibility that excessive local stress concentration is produced at the 1st story and that the structure collapses due to this stress concentration during strong earthquakes, if the structure is not designed under due consideration for structural irregularities on the whole structure.

Based on the Criterion on the Evaluation of Aseismic Safety of Existing Reinforced Concrete Buildings and on dynamic analyses, it was found that degree of aseismic safety of these buildings was very poor at especially the 1st story. In order to retrofit these buildings, preparation of additional resisting walls and wing walls, and also strengthening existing columns themselves using metal meshes were planned at the 1st story. First the method and details of the retrofitting applied to these buildings are introduced, and then, evaluation of structural characteristics of the buildings before and after the retrofitting, and finally, dynamic analyses to examine their aseismic safety during strong earthquakes are carried out in this paper.

1. INTRODUCTION

Buildings in which a volume of resisting walls at the 1st story is very poor compared with that at the other higher stories generally have a tendency to bring excessive local stress concentration to the soft 1st story during earthquakes. If these buildings are not designed under due consideration for structural irregularities along their heights, these buildings will collapse due to this stress concentration during strong earthquakes.

The authors had a chance to evaluate aseismic structural performance concerning this type of eleven existing R.C. housing structures in Tokyo. Based on the evaluation and dynamic analyses, ten of the eleven buildings were found to be unsafe against strong earthquakes. Retrofitting at the soft 1st story of these ten buildings was planned to improve aseismic capacity on the soft story. Then evaluation of aseismic structural performance and dynamic analyses of these buildings retrofitted were carried out to examine their aseismic safety as the whole building.

In this paper, the method and details of the retrofitting applied to these buildings are first introduced in Section 3 using typical two buildings, the Buildings-A and -B, which are four and six story reinforced concrete housing structures, respectively. Then, evaluation of structural characteristics of the buildings before and after the retrofitting and finally dynamic analyses to examine their aseismic safety during strong earthquakes are carried out in Sections 4 and 5, respectively.

Retrofitting all the eleven buildings was planned for only the soft 1st story because of difficulties to let the residentiary remove during retrofitting works, even if the upper stories also had poor aseismic capacity. The retrofitting at the 1st story was hence carefully carried out under due consideration concerning structural balance through all the stories so that new stress concentration was not developed at the other certain story.

For easy understanding, the Buildings-A and -B after their retrofitting will be distinguished as the Buildings-A_R and -B_R, respectively in the following sections.

2. OUTLINE OF BUILDINGS

The Building-A which was constructed in 1973, as shown in Figs.1 and 2 and Photo 1, is a four story housing with four and one spans in the longitudinal (X) and transverse (Y) directions, respectively, and is directly supported by continuous foundations. The Building-B which was constructed in 1968, as shown in Figs.3 and 4 and Photo 2, is a six story housing with five and one spans in the longitudinal and transverse directions, respectively, and is supported by R.C. piles with 17 meters in length.

Staircases are located at the both longitudinal ends of both the buildings. However volumes of resisting walls at the 1st story in both the directions are very poor compared with those at the other stories on both of the buildings. No fire and no major earthquakes have been reported in the history of these buildings.

3. METHOD AND DETAILS OF RETROFITTING

3.1 Aseismic Safety of the Buildings before Retrofitting

Aseismic safety of the Buildings-A and -B is discussed in detail in Section 4. Main points, which are common to both of the buildings, derived from the section are summarized as follows;

a. A volume of resisting walls at the 1st story is very poor compared with that at the other stories (which generally leads a soft first story type of buildings). Shear capacity of columns at the 1st story is not high enough compared with flexure capacity of the columns. Thinking collectively, degrees of aseismic safety at the 1st stories of these buildings are insufficient.

b. In the longitudinal direction at all the stories except the 1st story, there are many walls which were neglected when structural designs of these buildings were carried out. Therefore, actually, aseismic strengths of the stories of these buildings are comparatively high. However deformation capacity is not sufficient.

c. Volumes of resisting walls in the transverse direction at all the stories except the 1st story are sufficient enough and, consequently, degrees of aseismic safety at the stories are excellent.

3.2 Policies on Retrofitting

Policies on retrofitting the Buildings -A and -B were decided as follows after evaluation of their aseismic safety.

a. Retrofitting is restricted to be carried out at only the 1st story, because;
   i. These buildings are residential except the 1st story and removing the residentiary during retrofitting works is difficult.
   ii. Great improvement on aseismic safety of these buildings can be expected by retrofitting the 1st story.
   iii. There is at present no settled way for retrofitting effectively the 2nd story and higher in the longitudinal direction.

b. Retrofitting the 1st story is planned to be carried out considering balance on structural characteristics, such as aseismic strength and rigidity, through all the stories of the buildings.

3.3 Method of Retrofitting

The following items concerning retrofitting works were chosen to solve problems which were found in the Buildings-A and -B.

a. Additional resisting walls in the longitudinal (X) direction were prepared at the 1st story of these buildings so that a shape of rigidity distribution along the height of the buildings was improved and an aseismic strength at the 1st story was increased. Similarly, additional resisting walls in the transverse (Y) direction were prepared at the 1st story of the Building-B.

b. Wing walls which were adjoined to columns were prepared instead of preparing resisting walls without opening at the north side of these buildings so that a failure mechanism at the 2nd story, which was expected to be a flexure failure, was not changed to shearing one.

c. Columns located at the 1st story were planned to be strengthened with metal meshes so that shearing failure due to longitudinal horizontal forces and shearing-compression failure due to axial compression stress by transverse horizontal forces were not taken place at the columns which were disconnected with the additional resisting walls.

d. Slits with enough space were prepared at top and bottom of columns when metal meshes were put on the columns so that flexure failure was taken place ahead of shearing failure.

e. Mechanical anchors were prepared between a beam and an additional wall to connect with each other.

Arrangement of additional resisting walls and wing walls planned based on the items mentioned above are shown in Figs. 5 and 6.

3.4 Details of Retrofitting

Structural details were designed based on the Design Guide Lines for Aseismic Retrofitting of Existing Reinforced Concrete Buildings (2). Details on retrofitting columns are shown in Fig. 7 and those on additional resisting walls and wing walls are shown in Figs. 8a and 8b, respectively.

4. EVALUATION OF STRUCTURAL CHARACTERISTICS

4.1 Rigidity, Strength and Ductility

4.1.1 Rigidity

Longitudinal and transverse horizontal rigidities at each story of the Buildings-A and -B were evaluated considering flexure, shearing and rotating deformations based on elastic rigidities of resisting elements including walls with opening (which had been neglected in original structural design of these buildings). The rotating deformation was evaluated based on elastic deformations of soils or supporting piles. Horizontal rigidity distributions evaluated based on the criteria described here are shown in Fig. 9. The main features in Fig. 9 are as follows for both of these buildings;

i. Longitudinal horizontal rigidities of walls with opening at the 2nd story and higher are very large compared with those of columns, so that there is a big difference in the rigidities at the 1st story and the other stories.

ii. Transverse horizontal rigidities at the 1st story are somewhat smaller than those at the other stories due to a smaller volume of resisting walls at the 1st story.

iii. Transverse horizontal rigidities through all the stories are small compared with corresponding longitudinal ones because of large rotating deformations in the transverse direction.

4.1.2 Strength

Horizontal capacity at each story of these buildings, in both the longitudinal and transverse directions, was evaluated considering flexure, shearing and rotating capacity of columns, beams and walls based on both the Criterion on the Evaluation of Aseismic Safety of Existing Reinforced Concrete Buildings (1) and the Design Guide Lines for Aseismic Retrofitting of Existing Reinforced Concrete Buildings (2). The formula to evaluate the (equivalent) capacity is briefly introduced in Section 4.1.4. The results are shown in Fig.10 in the form of shear coefficients (called Ce-Index).

4.1.3 Ductility

Ductilities at each story of buildings, in both the longitudinal and transverse directions, were evaluated in the form of the equivalent Fe-Index which was synthetically obtained using F-Index values of all resisting elements at the story based on a method described in the literatures (1,2). Figure 11 shows the final results in which main features are;

i. Values of the Fe-Index at the 1st story of the Building-A in both the directions and the Building-B in the longitudinal direction are smaller compared with those at the other stories.

ii. Values of the Fe-Index at the 2nd story and higher of the Building-B in the transverse direction are large. Those values were obtained from an overturning-type failure mechanism.

4.1.4 Equivalent Strength and Ductility

The method to evaluate horizontal capacity of a building has not yet established in case the building is composed of both ductile and brittle resisting elements. For this purpose, a method has been proposed in the literature (1) in which the Basic Structural Performance Index, Eo, is used to evaluate overall capacity of a building having ductile and brittle resisting elements which have independent strengths and ductilities. Using the Eo-Index, the following ways to evaluate horizontal capacities and ductilities at each story of the Buildings-A and -B were prepared in this paper, that is;

i. A larger value in the values, Ce_1 and Ce_2, which are obtained in the following two items, ii and iii, are chosen as an equivalent shear coefficient, Ce, at each story of a building. On the other hand, a smaller value in the values, Fe_1 and Fe_2, which also are obtained in the following items are chosen as an equivalent Ductility Index, Fe, at each story of a building.

ii. The values of the Indexes Ce_1 and Fe_1 are evaluated as follows in case a building is mainly composed of ductile resisting elements.

$$Ce_1 = \frac{F_T - F_S}{W} \qquad (1)$$

$$Fe_1 = \frac{Eo}{Ce_1} \cdot \frac{n + i}{n + 1}$$

iii. The values of the Indexes Ce_2 and Fe_2 are evaluated as follows in case a building is mainly composed of brittle resisting elements.

$$Ce_2 = \frac{F_S + 0.7\,F_B}{W} \qquad (2)$$

$$Fe_2 = \frac{Eo}{Ce_2} \cdot \frac{n+i}{n+1}$$

The symbols, F_T, F_B, F_S, W, n and i, in the above items, ii and iii, denote the following, respectively, that is;

F_T: total strength of all resisting elements

F_B: total strength of bending-type resisting elements

F_S: total strength of shearing-type resisting elements

W: sum of weights from the i-th story to the top story

n: number of stories

and

i: story number counted from the top story to lower

4.2 Seismic Performance of the Buildings

According to the Criterion (1), overall aseismic safety of buildings is evaluated based on the Aseismic Structure Index, I_S, that is;

$$I_S = Eo \cdot G \cdot S_D \cdot T \qquad (3)$$

where

Eo: Aseismic Sub-Index of Basic Structural Performance

G: Aseismic Sub-Index of Ground Motion

S_D: Aseismic Sub-Index of Structural Profile

and

T: Aseismic Sub-Index of Time-Dependent Deterioration

Values of the Index, I_S, of the Buildings-A and -B, which are evaluated based on Eq. 3, are shown in Fig. 12. In this Figure, the value of the symbol I_{SO} is a standard value which was proposed as a threshold for judging aseismic safety of buildings subjected to ground mothions which are as strong as the Tokachi-Oki Earthquake in 1968. As seen in Fig. 12, the I_S values at the 1st story of the Buildings-A_R and-B_R which were strengthened were greatly improved, however, the values at the 2nd to 5th stories of the Building-B (and also the Building-B_R) were smaller than the standard value symbolized with I_{SO}.

4.3 Effects of Retrofitting

The Building-A

In the longitudinal (X) direction, a resisting wall without opening and four wing walls were newly prepared at the 1st story, and also all the columns located at the 1st story were reinforced using metal meshes to prevent shearing failure. Consequently, the Ce-Index value at the 1st story was doubled from 0.4 to 0.8. However, the Fe-Index value at the 1st story was not changed in spite of improvement on ductilities of columns, for a failure mechanism of the additional resisting wall was shearing failure one.

In the transverse (Y) direction, no additional resisting walls were prepared because of existence of two resisting walls. The Ce-Index values in this case were slightly changed. However the Fe-Index value at the 1st story was greatly increased due to large improvement on ductilities of columns by retrofitting with metal meshes.

Finally, the I_S values at all the stories topped the standard value I_{SO}.

The Building-B

In the longitudinal (X) direction, two resisting walls without opening and two wing walls were newly prepared at the 1st story, and also all the columns located at the 1st story were reinforced for shearing failure. After the retrofitting the Ce-Index value was approximately doubled from 0.3 to 0.6. On the other hand, the Fe-Index value was slightly increased, because some additional walls had shearing failure mechanisms.

In the transverse (Y) direction, two resisting walls were newly prepared at the 1st story in addition to two existing walls. The Ce-Index value was changed by retrofitting from 0.3 to 0.4. The reason why the value was not greatly improved by retrofitting is because of overturning-type failure mechanisms on all the resisting walls. On the contrary, the Fe-Index value was increased up to 3.0, for failure mechanisms on all resisting walls were changed to ductile failure ones and also ductilities on all the columns were greatly improved by retrofitting with metal meshes.

Finally, the I_S values at the 1st story topped the standard value I_{SO}. However the values at the 2nd to 5th stories in the longitudinal direction were smaller than the value of I_{SO}. At these stories, only S_D - Index values were improved a little bit. To confirm aseismic safety of this building, especially in the longitudinal direction, dynamic analyses are carried out in the following section.

5. DYNAMIC ANALYSIS

5.1 Outline of Analysis

Structures

Dynamic analyses of the Buildings-A and -B, which were idealized as both a linear and a bi-linear lamped mass system supported on a rigid base, were carried out. The damping constant, h, taken through the analyses was 5% as a fraction of critical damping.

The slope angle of the second line in bi-linear hysteresis curves was evaluated as the value of 10% of the initial slope angle.

Earthquake Ground Motions

Two accelerograms, that is the El Centro Earthquake (NS-Component) recorded during the Imperial Valley Earthquake in 1940 and the Hachinohe Earthquake (EW-Component) recorded during the Tokachi-Oki Earthquake in 1968, were applied to the dynamic analyses. The maximum accelerations of these accelerograms were normalized to 225 gals and 450 gals for a linear and a bi-linear response analysis, respectively. The magnitudes of the maximum accelerations were determined based mainly on statistic analyses concerning seismic intensity expected in future earthquakes and on experimental studies concerning the maximum ground motions. The two magnitudes, 225 gals and 450 gals, correspond to those for the return period of 25 and 100 years, respectively, in Tokyo.

Time histories and acceleration response spectra of these two accelerograms are shown in Figs. 13a, 13b, 14a and 14b in order.

Natural Periods and Modes

Natural periods of the buildings before and after their retrofitting are listed in Table 1. The fundamental natural periods generally become shorter by the retrofitting. Natural modes of the buildings before and after their retrofitting are shown in Figs. 15a and 15b. Discontinuity of modes can be seen, in these figures, at the 1st story of the buildings before retrofitting in especially the longitudinal (X) direction. It can be recognized, however, that after the retrofitting the discontinuity is almost disappeared and that mode shapes become smooth through all the stories.

5.2 The Maximum Responses

The Building-A

The maximum values of linear responses, such as accelerations, shear forces, shear coefficients, deflections relative to the base, story deflections and angles of deflections are shown in Figs. 16 and 17 for the Buildings -A and -A_R in both the directions, respectively, and the corresponding values of non-linear responses including ductility factors are shown in Figs. 18 and 19. Differences in these responses between the Building-A (before retrofitting) and the Building-A_R (after retrofitting) are summarized as follows;

(1) Responses in the longitudinal (X) direction

- i. Deflection distribution discontinuity at the 1st story was greatly improved after the retrofitting in both the responses to 225 gal and 450 gal ground motions.
- ii. For 450 gal ground motions, ductility factors at the 1st story were decreased from 2 - 3.5 to 1.0 after the retrofitting. On the contrary, the corresponding ductility factors at the 2nd and the 3rd story were increased from 1 - 2 to 2 - 3.
- iii. However, the values of story deflections at the 2nd and the 3rd story of the Building-A_R are still less than 0.5 cm which corresponds to 1/600 in the angle of deflection.

(2) Responses in the transverse (Y) direction

- i. Differences in responses between the Buildings-A and -A_R are small. This is a natural result, because no additional walls were planned in this direction.
- ii. For 450 gal ground motions, ductility factors at the 1st story are in the range 2 - 3 which corresponds to 1/200 - 1/150 in the angle of deflection.
- iii. The corresponding values at the 2nd story and higher are all less than half of the values at the 1st story.

It can be recognized under synthetical judgement concerning above items that the Building-A has sufficient aseismic safety in both the directions against even 450 gal ground motions after the retrofitting was carried out.

The Building-B

The maximum values of linear responses are shown in Figs. 20 and 21 for the Buildings-B and -B_R in both the directions, respectively, and the corresponding values of non-linear responses are shown in Figs. 22 and 23. Differences in the responses between the Buildings-B and B_R are summarized as follows;

(1) Responses in the longitudinal (X) direction

i. Deflection distribution discontinuity at the 1st story was greatly improved after the retrofitting in both the responses to 225 gal and 450 gal ground motions.

ii. For 450 gal ground motions, ductility factors at the 1st story were decreased from 4 - 8 to 1.0 after the retrofitting. On the contrary, the corresponding ductility factors at the 2nd to the 4th stories, after the retrofitting, were almost doubled to the range of 3 - 6 from 1.5 - 4.

iii. However, the values of story deflections at these stories, the 2nd to the 4th stories, of the Building -B_R are still less than 0.8 cm which approximately corresponds to 1/400 in the angle of deflection.

(2) Responses in the transverse (Y) direction

i. For 225 gal ground motions, the maximum story deflections through all the stories, which were produced at the 1st story, were decreased from 1.2 - 1.7 cm (1/300 - 1/200 in the angle of deflection) to 0.8 cm (1/400) after the retrofitting.

ii For 450 gal ground motions, large discontinuity on response distribution was produced at the 1st story of the Building-B, and this discontinuity was not greatly improved on the Building -B_R.

iii The values of the corresponding story deflections at the 1st story of the Building-B_R are in the range of 3 to 4 cm which approximately corresponds to 1/150 - 1/100 in the angle of deflection.

It can be recognized under synthetical judgment concerning above items that the Building-B has sufficient aseismic safety in the longitudinal direction against even 450 gal ground motions, after the retrofitting was carried out. However, in the transverse direction, somewhat large story deflections are still expected at the 1st story against 450 gal ground motions, even if the retrofitting was carried out. Strengthening the 1st story in this direction a little more is desirable.

6. CONCLUSIONS

Investigation on aseismic safety and retrofitting design of two typical existing medium-rise R.C. housing structures in Tokyo which have soft 1st stories was introduced.

The (four story) Building-A in the longitudinal (X) direction had extremely small aseismic capacity at the 1st story and had enough capacity at the other stories. Aseismic capacity in the transverse (Y) direction of the building was poor at only the 1st story. It was confirmed after the investigation that retrofitting of the Building-A gave sufficient aseismic capacity to the 1st story and also to the whole structure.

On the other hand, the (six story) Building-B in the longitudinal direction also had extremely small aseismic capacity at the 1st story and had poor aseismic capacity at the other stories except the top story. Aseismic capacity in the transverse direction of this building was poor at only the 1st story. Due to poor capacity through all the stories except the top story in the longitudinal direction, retrofitting is needed not only for the 1st story, but also for the other stories having the poor capacity. However buildings under investigation are residential except the 1st story. Due to difficulties to let the residentiary remove during retrofitting works, retrofitting had to be planned for only the soft 1st story even if the upper stories also had poor aseismic capacity. Unbalance on structural characteristics through a structure gives local stress concentration and forces the structure to collapse during an strong earthquake. Therefore retrofitting on the soft 1st story must be carefully carried out under consideration about structural balance through all the stories of buildings.

Retrofitting concerning the 1st story of the Building-B was planned based on the consideration about the structural balance mentioned above. It was confirmed based on the Third Evaluation Method in the Criterion (1) and on dynamic analyses that aseismic characteristics of the 1st story of this building in the longitudinal direction was greatly improved. On the contrary, the maximum story deflections at the 2nd to the 4th stories, when subjected to 450 gal ground motions, were almost doubled after the retrofitting, however, the values of the deflections were less than 0.8 cm which approximately corresponds to 1/400 in the angle of deflection. Even if ductility factors at these stories were increased from 1.5 - 4 to 3 - 6 in this case, the magnitude of the angle of deflection, 1/400, is very small compared with a value of a limit angle of deflection which will restrain failures on the resisting elements having some amount of reinforcing bars which is needed under regulations concerned. Therefore, the Building-B in the longitudinal direction is considered to be safe through all the stories against strong earthquakes.

On the other hand, aseismic characteristics in the transverse direction of the Building-B was not greatly improved even after the retrofitting. The values of story deflections at the 1st story in this case were in the range of 3 - 4 cm (1/150 - 1/100 in the angle of deflection). Strengthening the 1st story in this direction a little more is desirable in dynamic point of view. However, the Aseismic Structure Index, I_S, at the 1st story of this building in this direction is,as shown in Fig. 12, considerably greater than the standard value, I_{SO}, which was proposed as a threshold for judging aseismic safety of buildings subjected to ground motions which are as strong as the Tokachi-Oki Earthquake in 1968. So no major critical damages are expected on the Building-B in the transverse direction under the retrofitting considered in this paper.

As mentioned above, aseismic capacity at a certain story must, if it is poor, be carefully improved under consideration concerning structural balance through all the stories of a building. Strengthening excessively a certain story having poor capacity without due consideration yields a possibility that stress concentration is newly developed at the other certain story, and hence that no contribution is expected for improving entire aseismic safety of a building.

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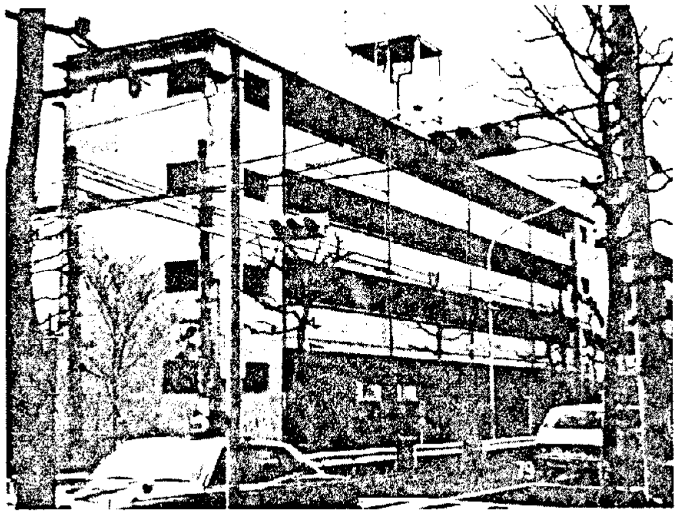


Photo 1   Building-A

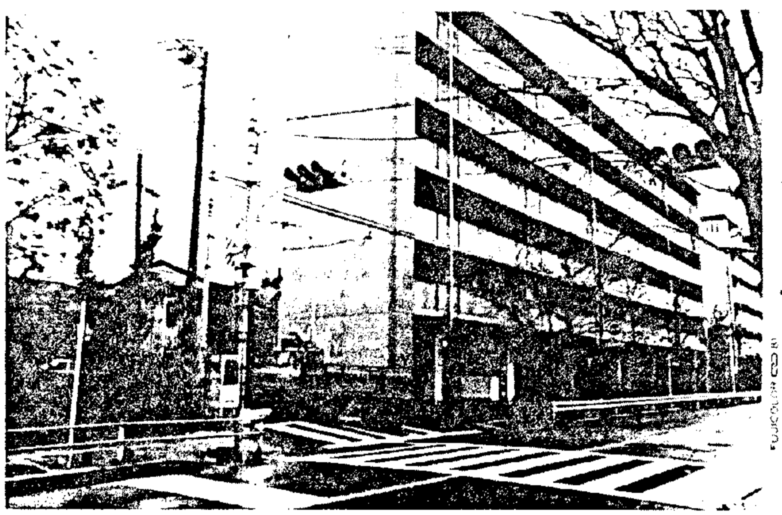


Photo 2   Building-B

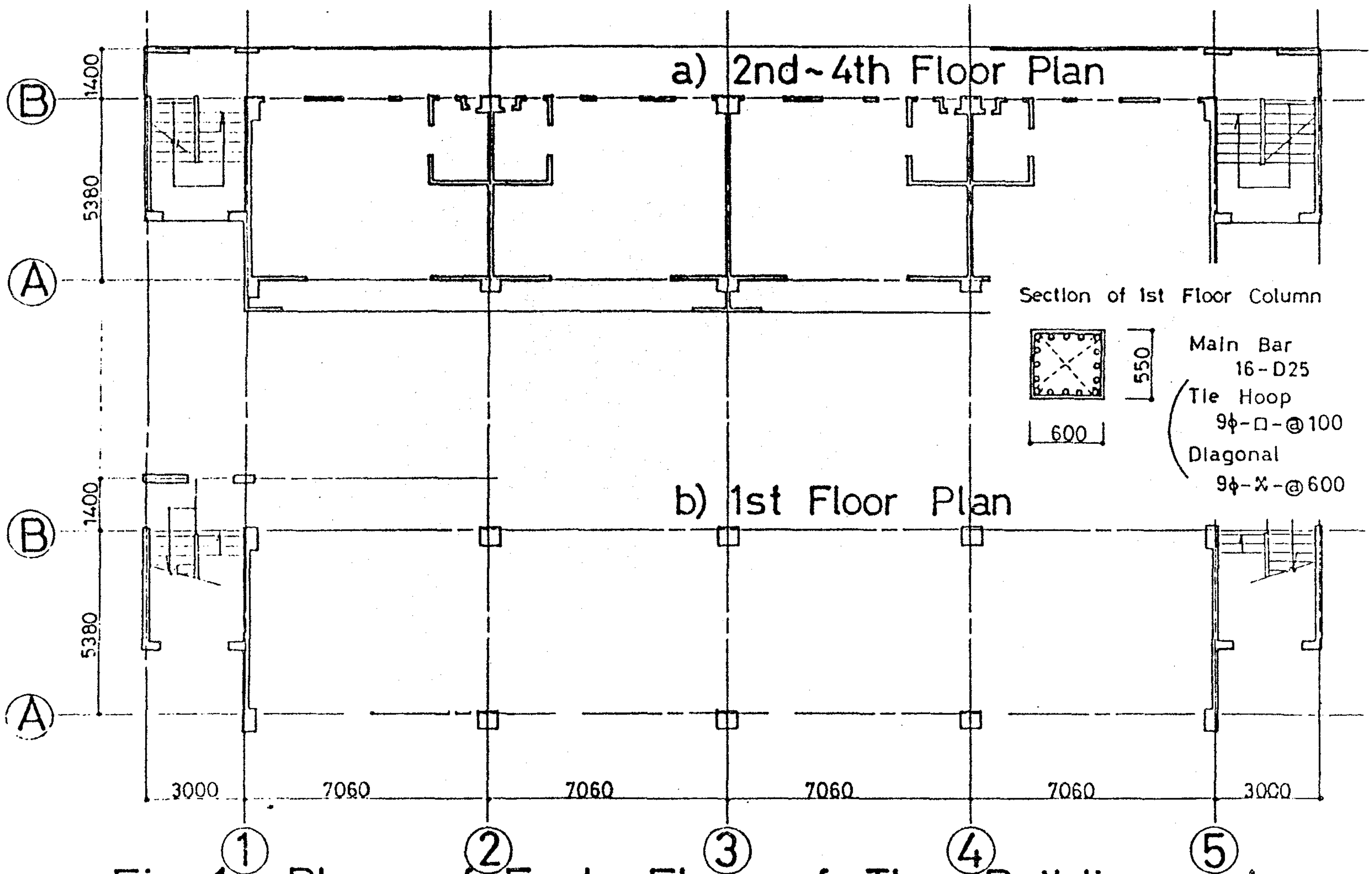
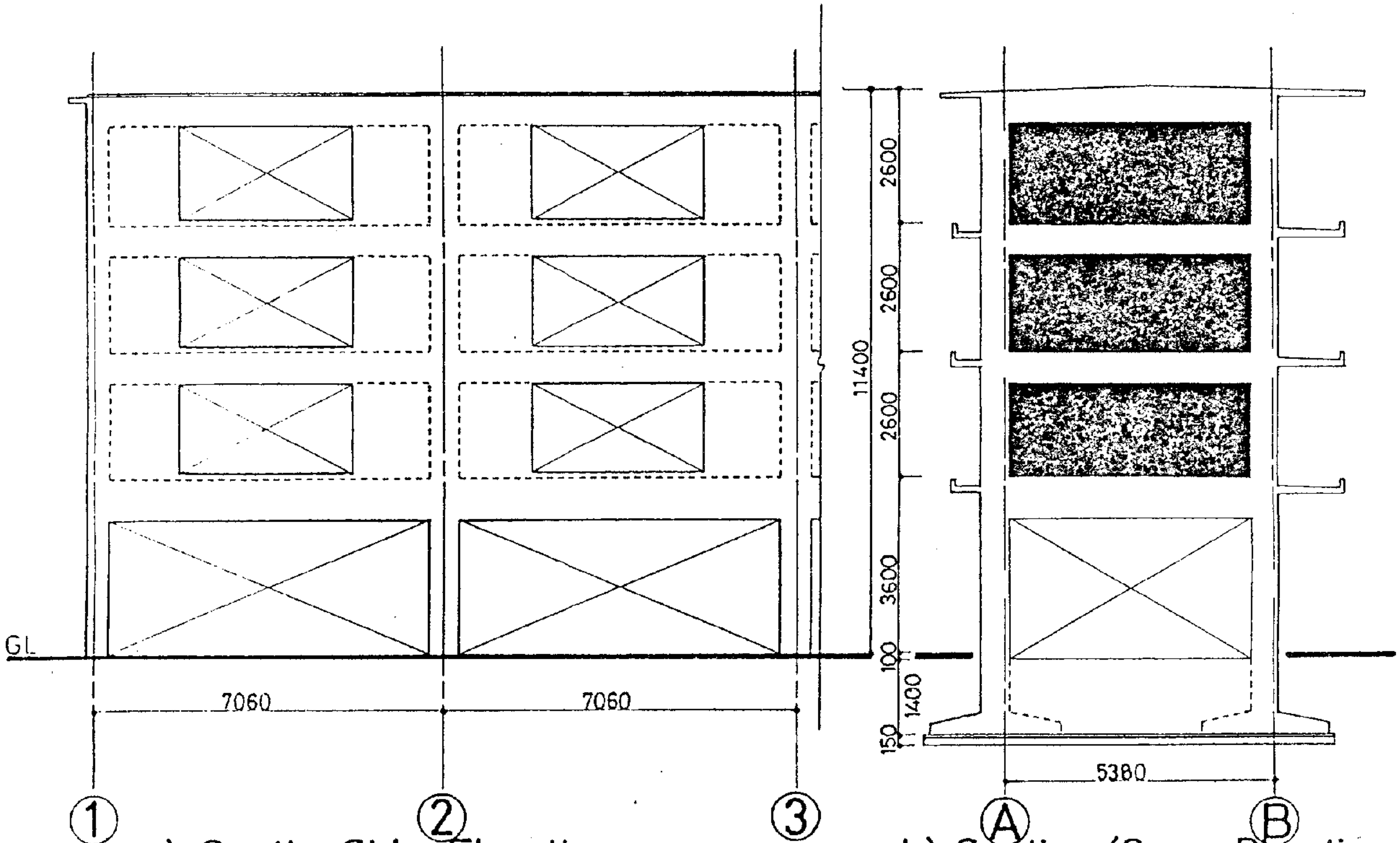


Fig. 1 Plans of Each Floor of The Building — A



a) South Side Elevation

b) Section (Span Direction)

Fig. 2 Elevation and Section of The Building — A

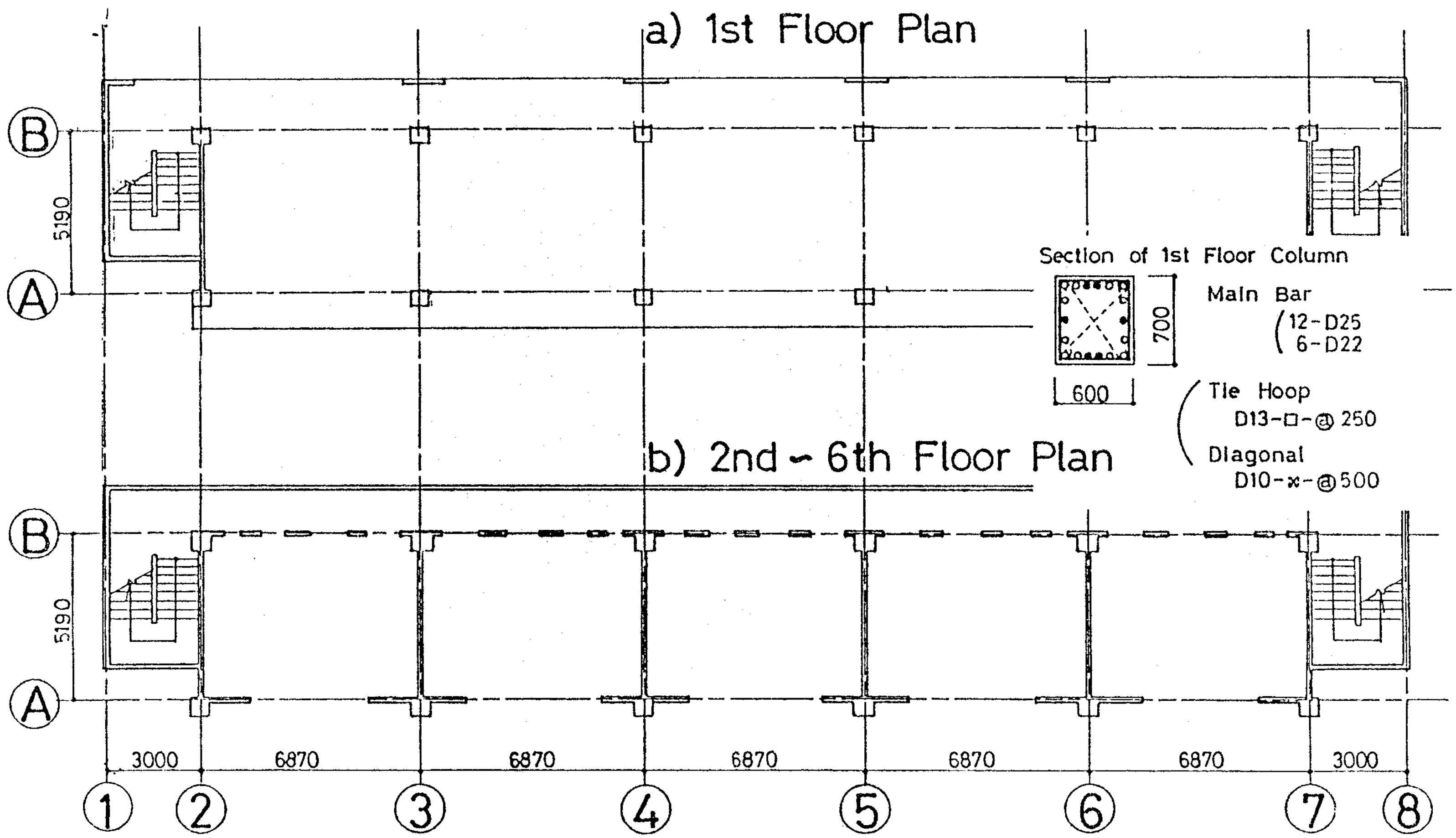


Fig. 3 Plans of Each Floor of The Building — B

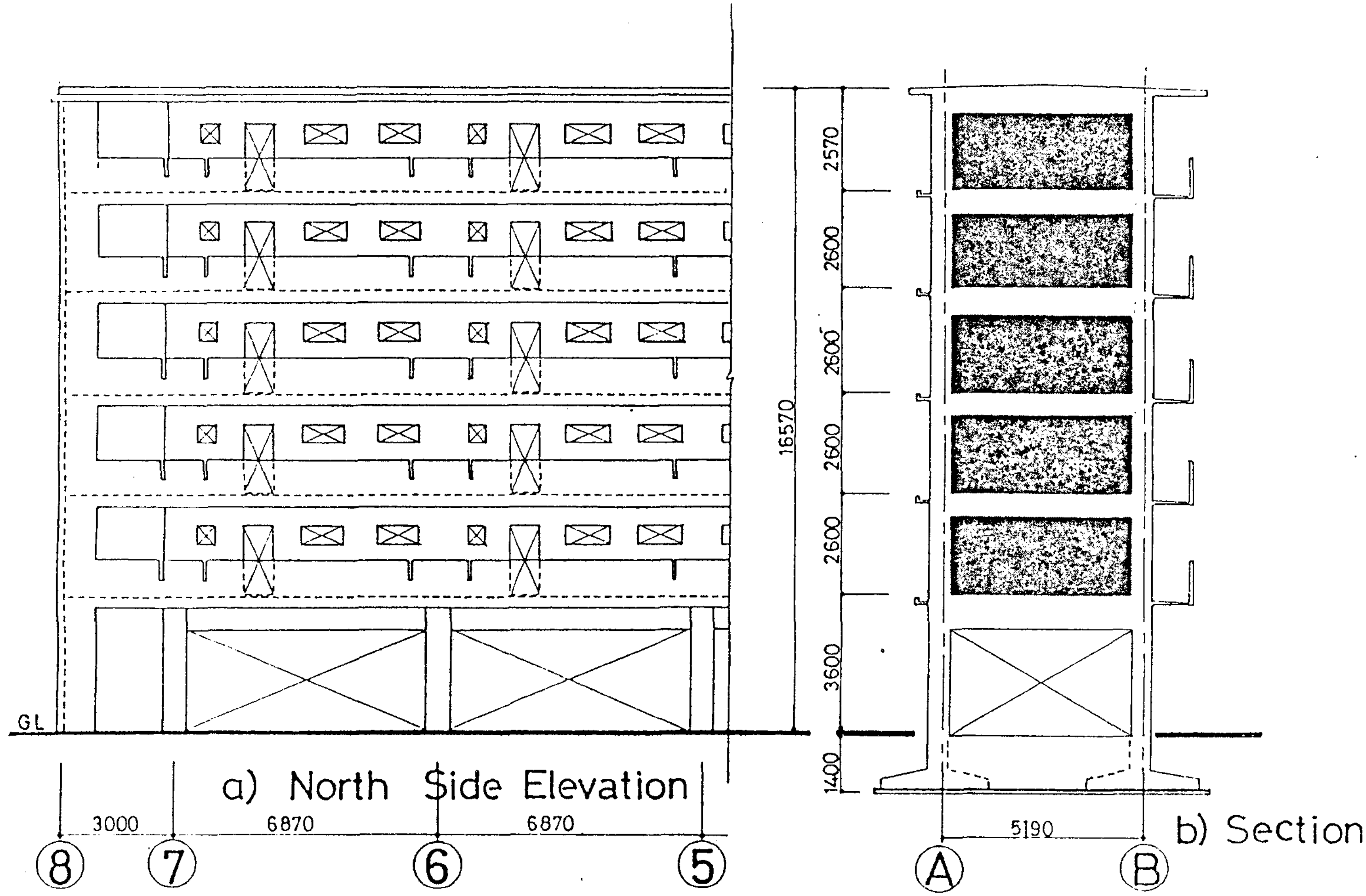


Fig. 4 Elevation and Section of The Building — B

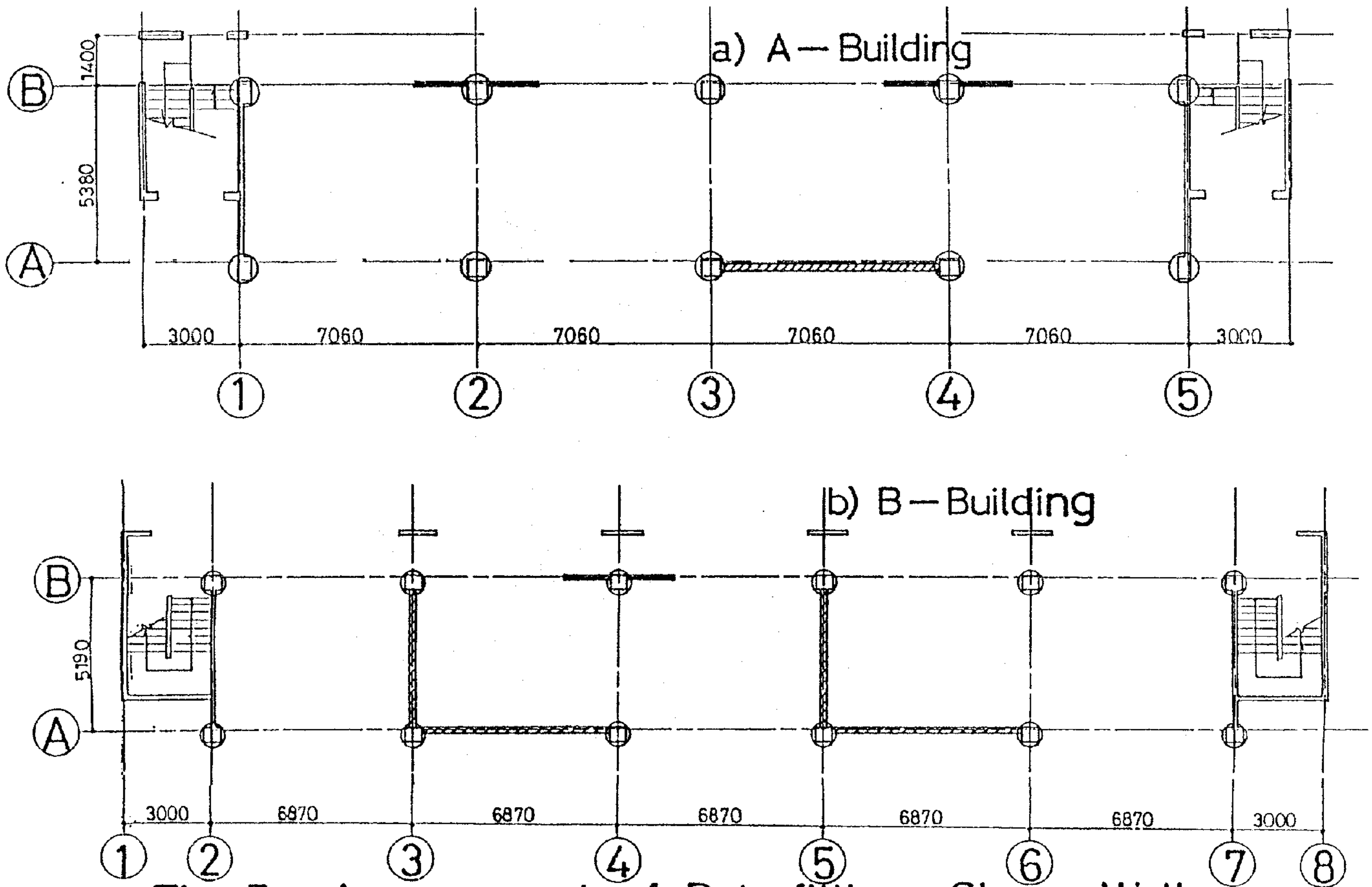
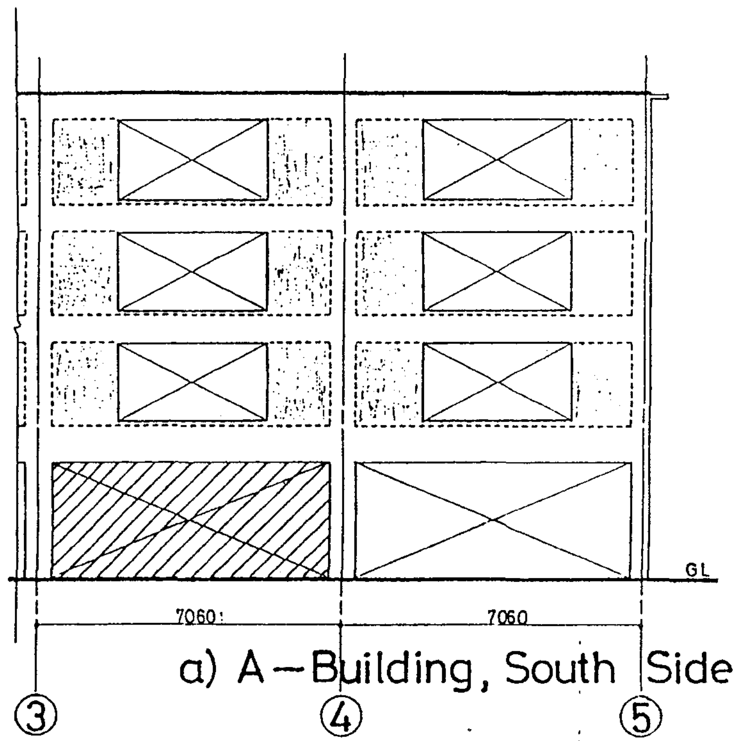
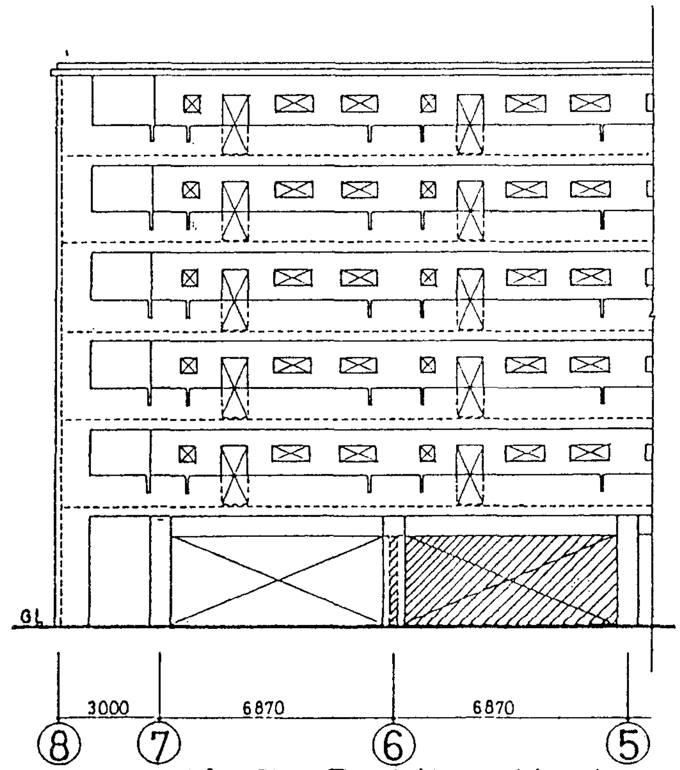


Fig. 5 Arrangement of Retrofitting Shear Walls and Wing Walls in Plans



a) A—Building, South Side



b) B—Building, North Side

Fig. 6 Arrangement of Retrofitting Shear Walls and Wing Walls in Structural Plans

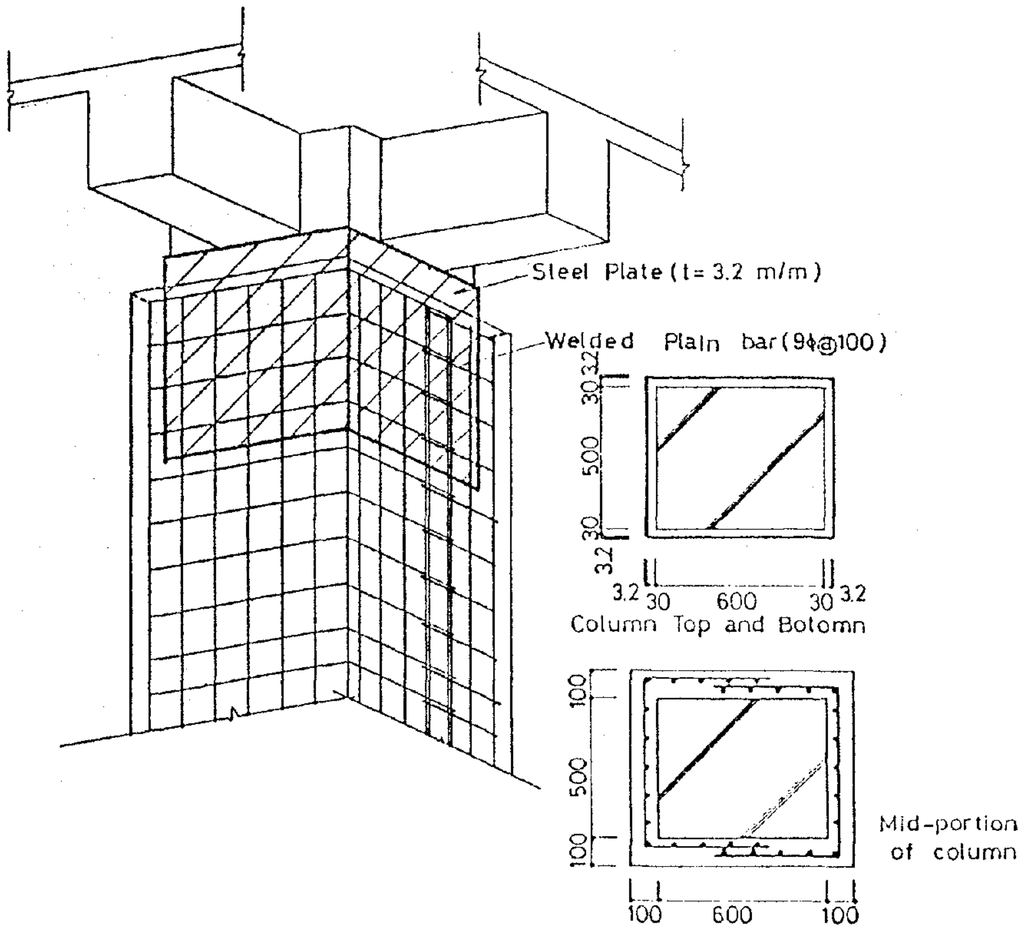


Fig. 7 Details of Retrofitting Column

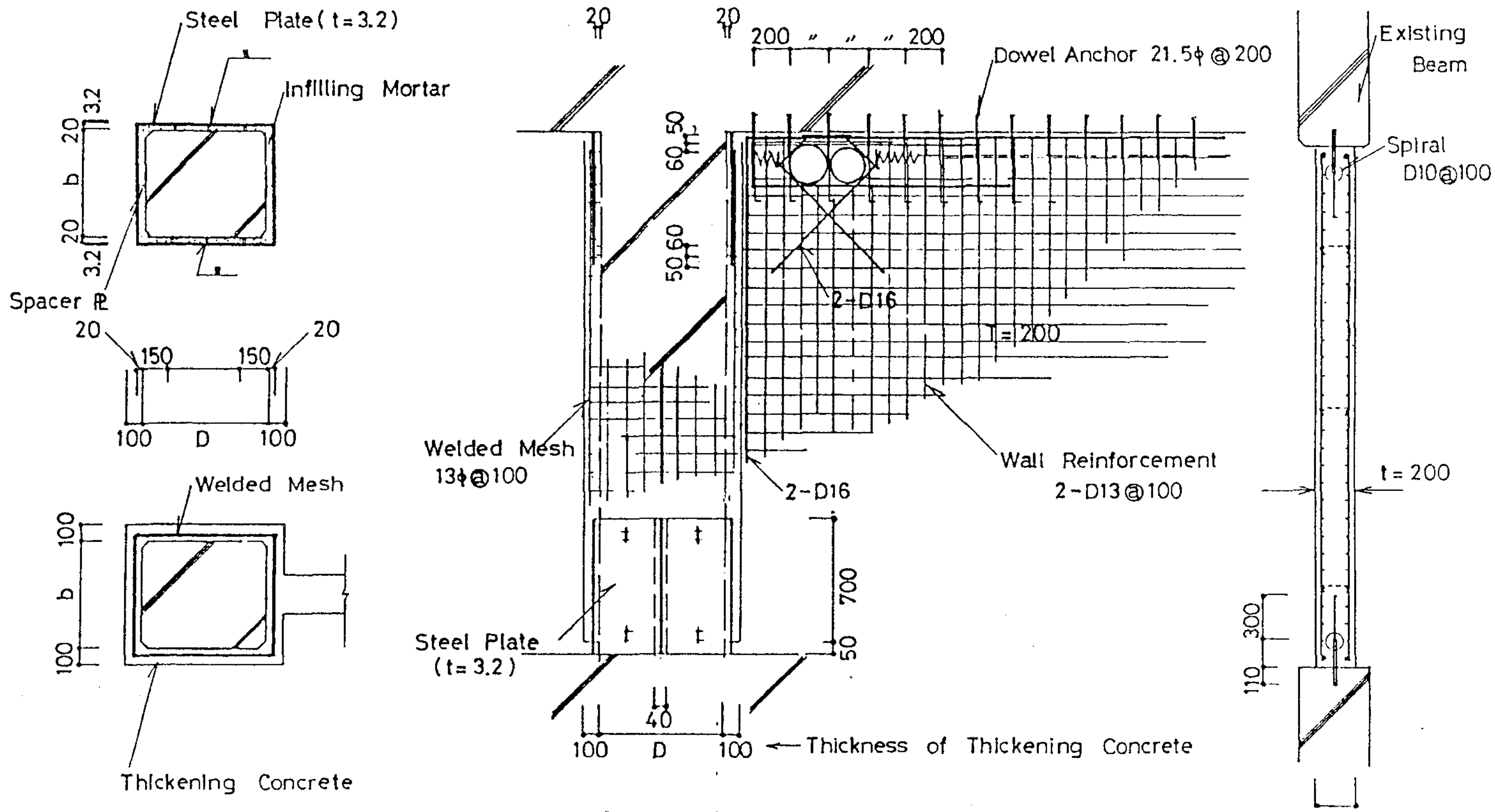


Fig. 8a Details of Retrofitting Shear Wall
(Unit mm)

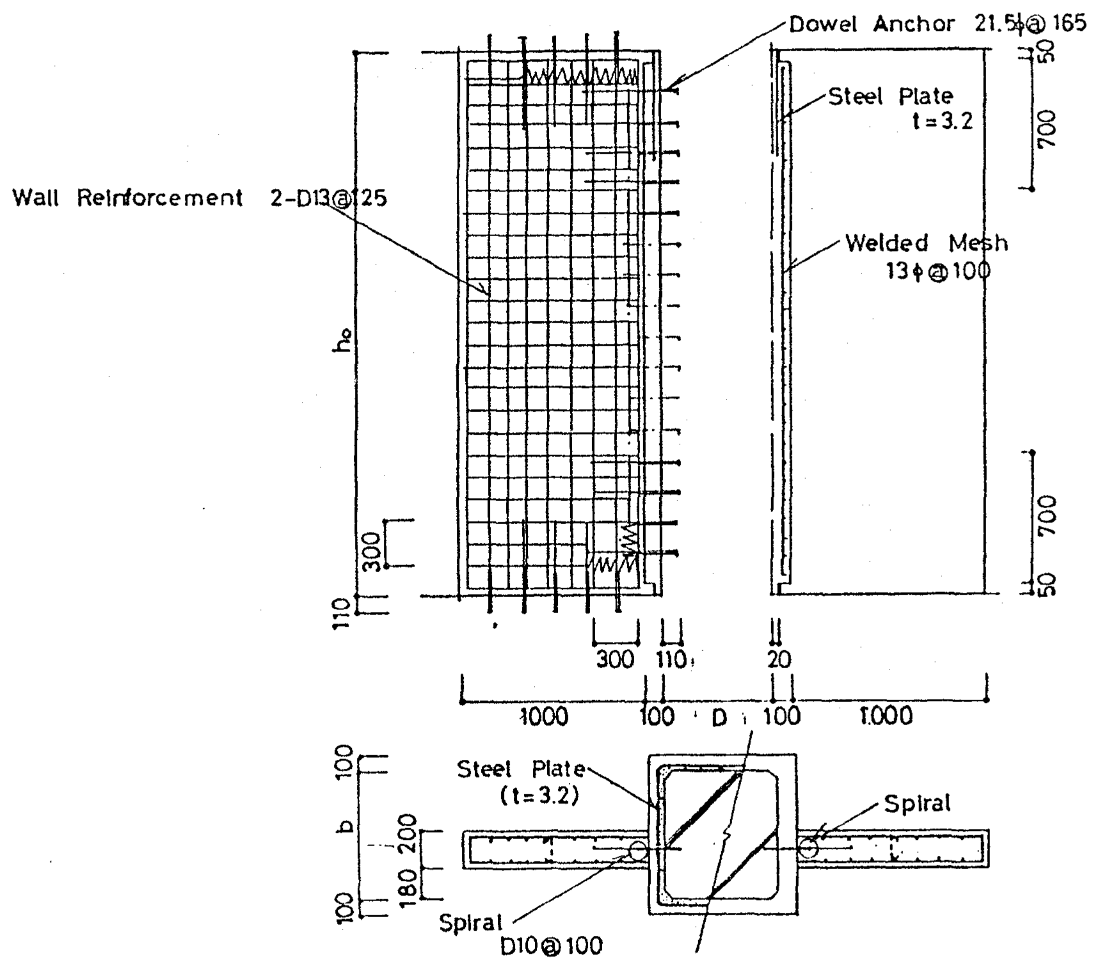
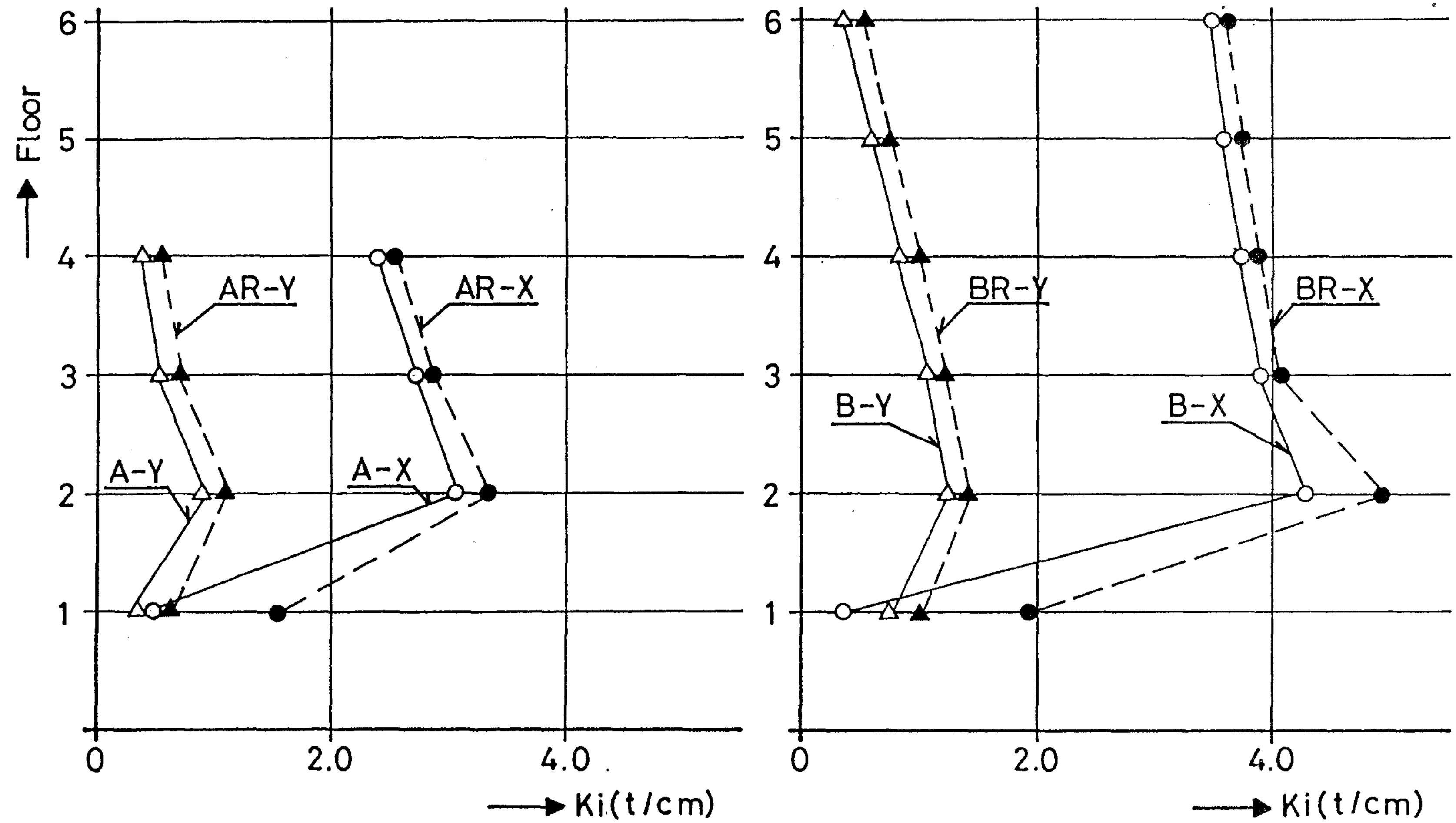


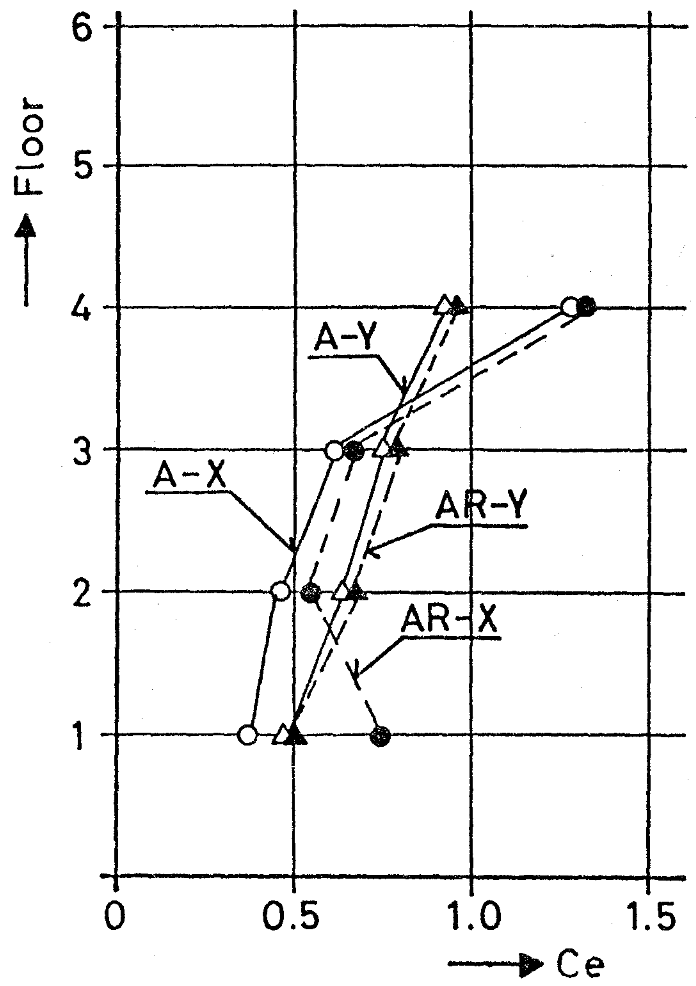
Fig. 8b Details of Retrofitting Wing Wall(Unit mm)



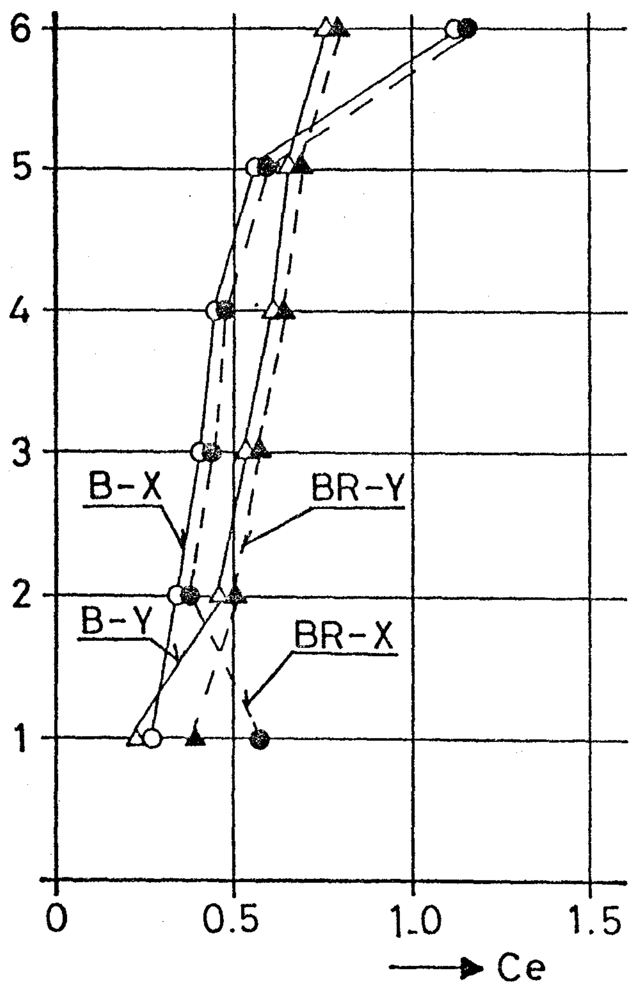
a) Building A,AR

b) Building B,BR

Fig.9 Horizontal Rigidity Distribution Before and After Retrofitting

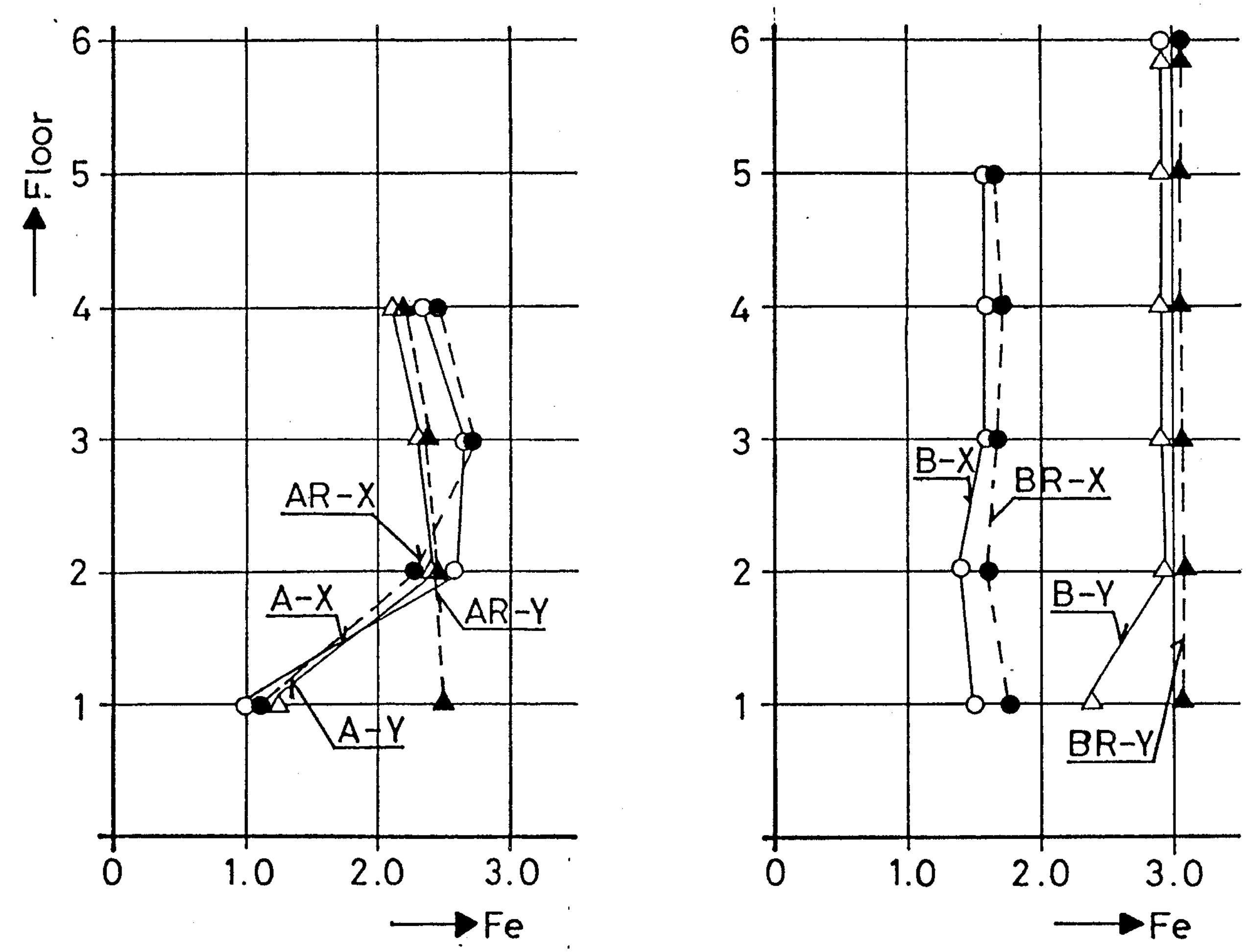


a) Buildings A,AR



b) Building B,BR

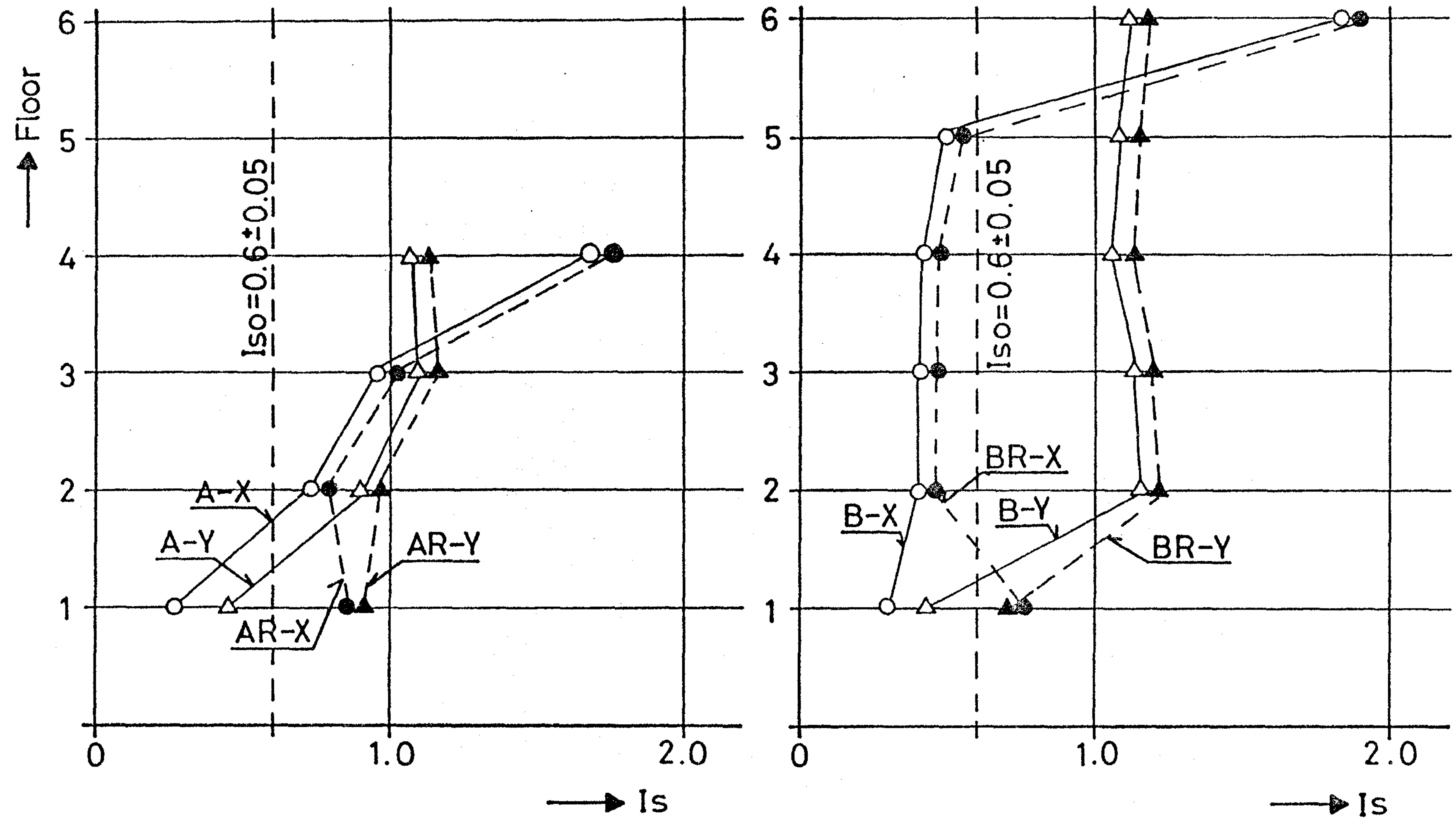
Fig. 10 Horizontal Shear Capacity Coefficient Distribution Before and After Retrofitting



a) Building A,AR

b) Building B,BR

Fig.11 Distribution of F-Indexes Before and After Retrofitting



a) Building A,AR

b) Building B,BR

Fig.12 Distribution of Calculated Aseismic Structure Index Is

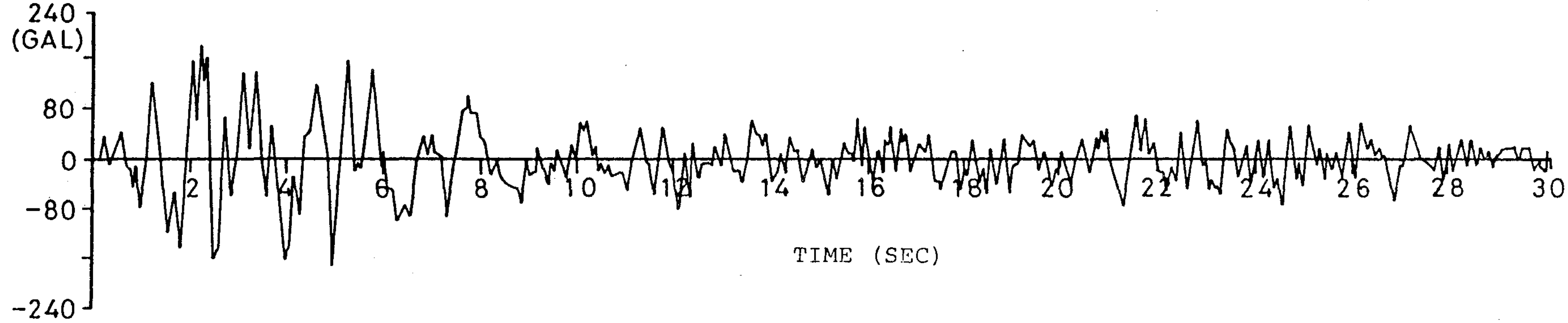


Fig.13a Time History of Imput Accelerogram

(Hachinohe 1968,EW)

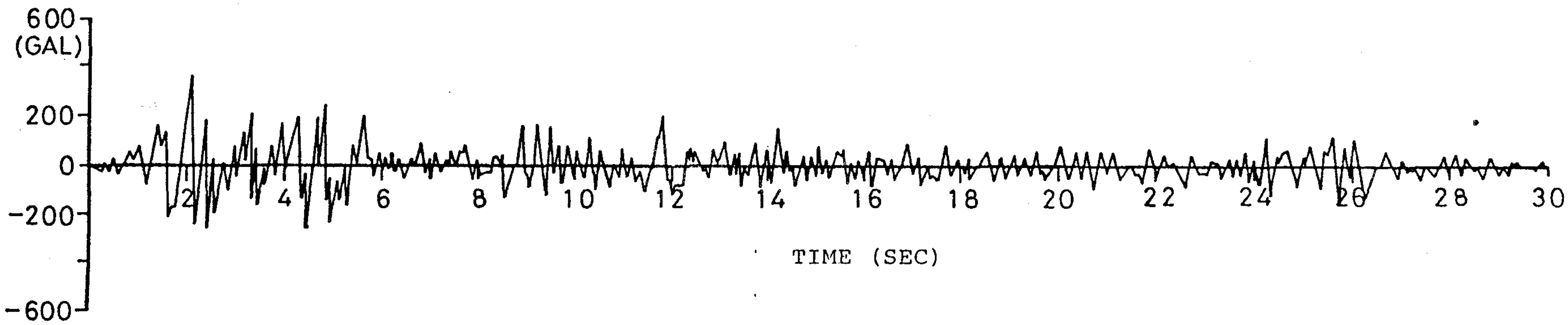


Fig.13b Time History of Imput Accelerogram

(El Centro 1940, NS)

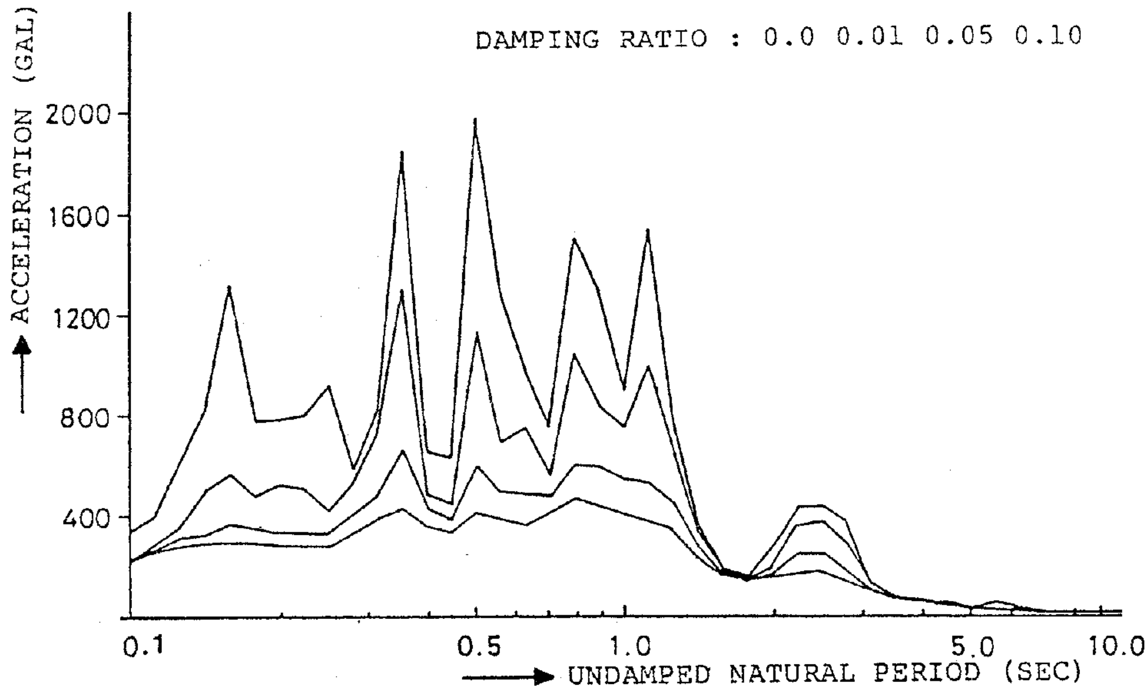


Fig.14a Acceleration Response Spectra (Hachinohe 1968, EW)

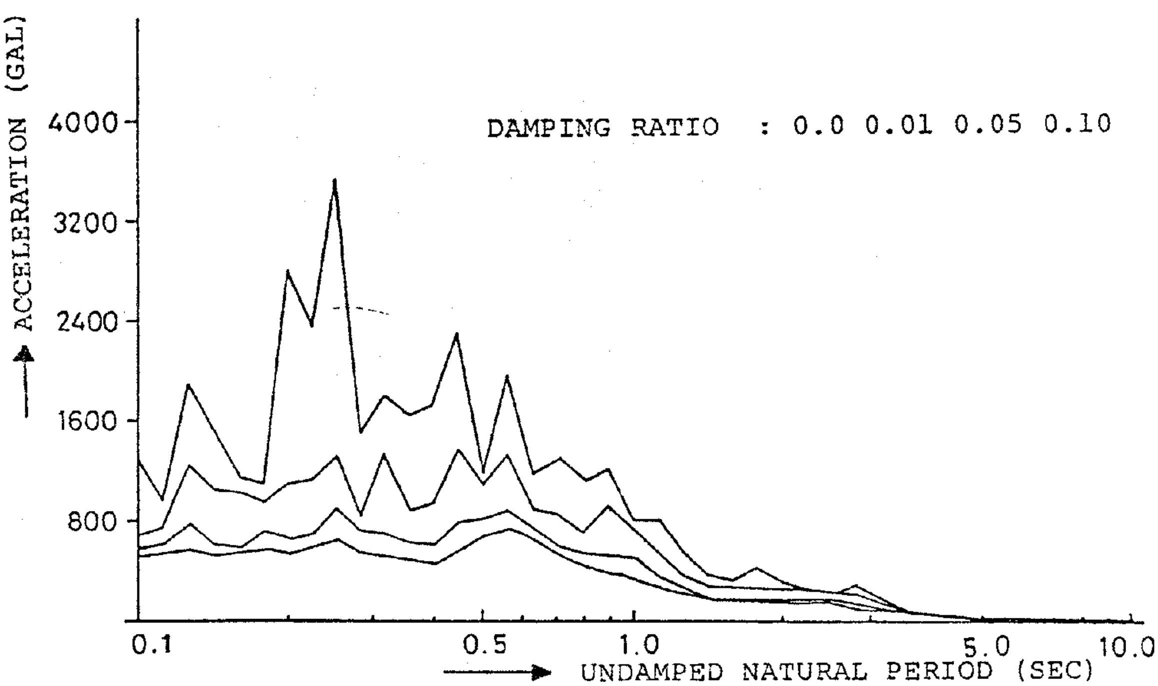
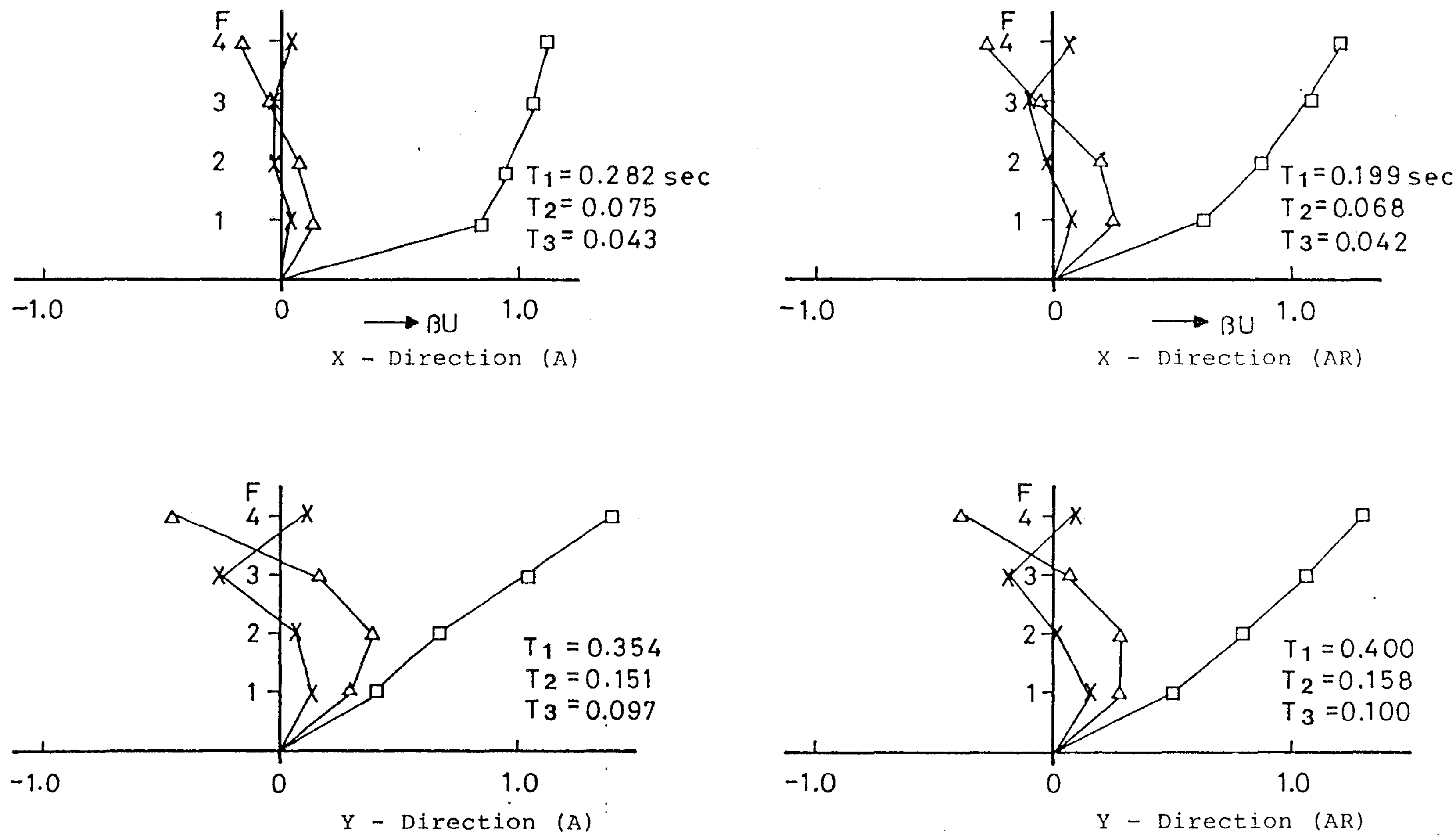
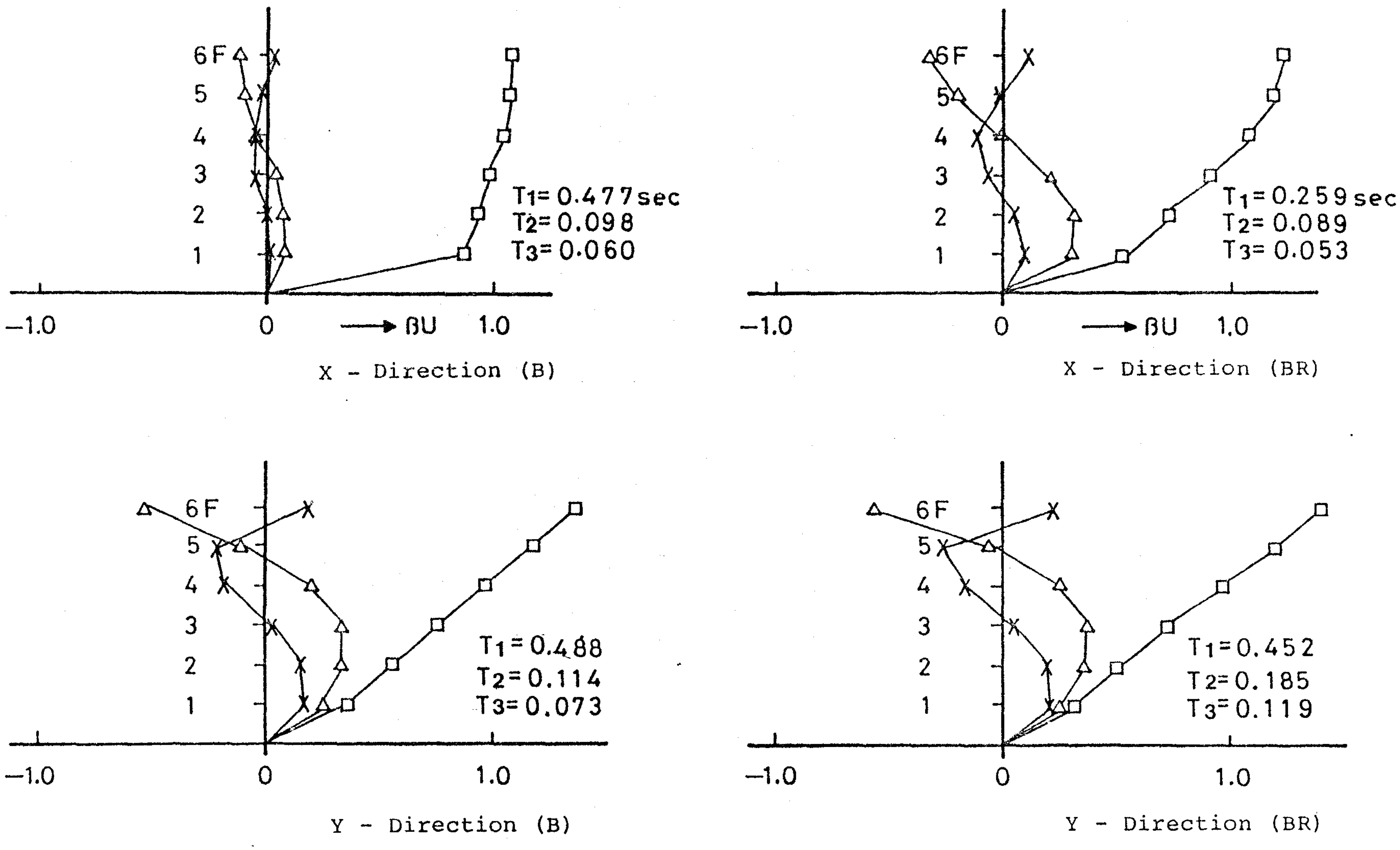


Fig.14b Acceleration Response Spectra (El Centro 1940, NS)



Before Retrofitting (A) After Retrofitting (AR)

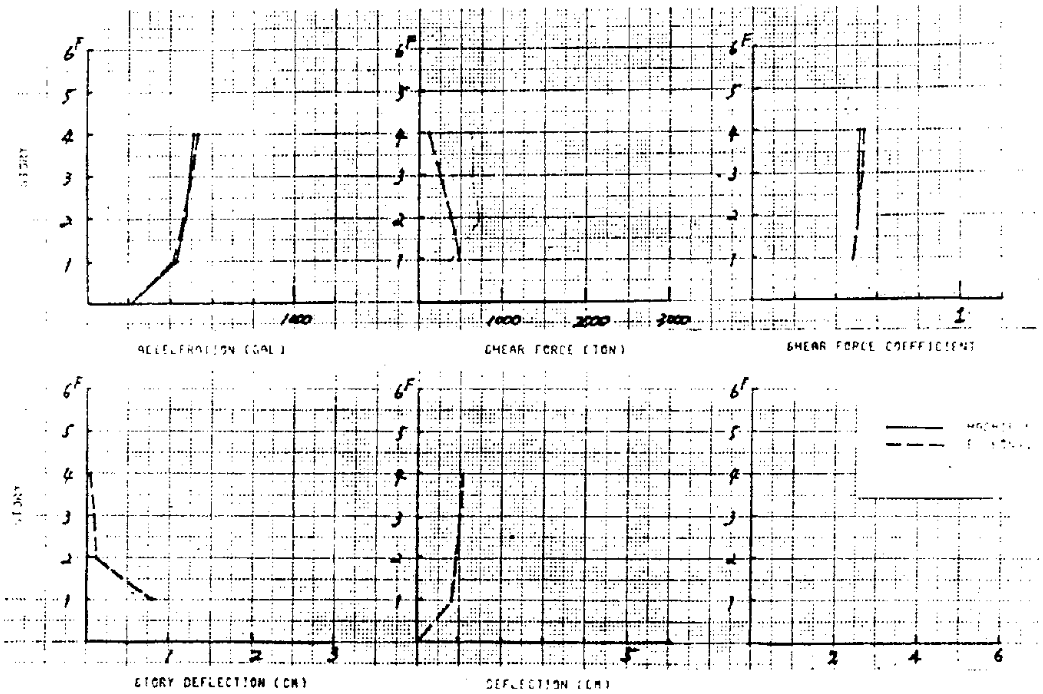
Fig. 15a Exciting Functions of the Buildings A and AR



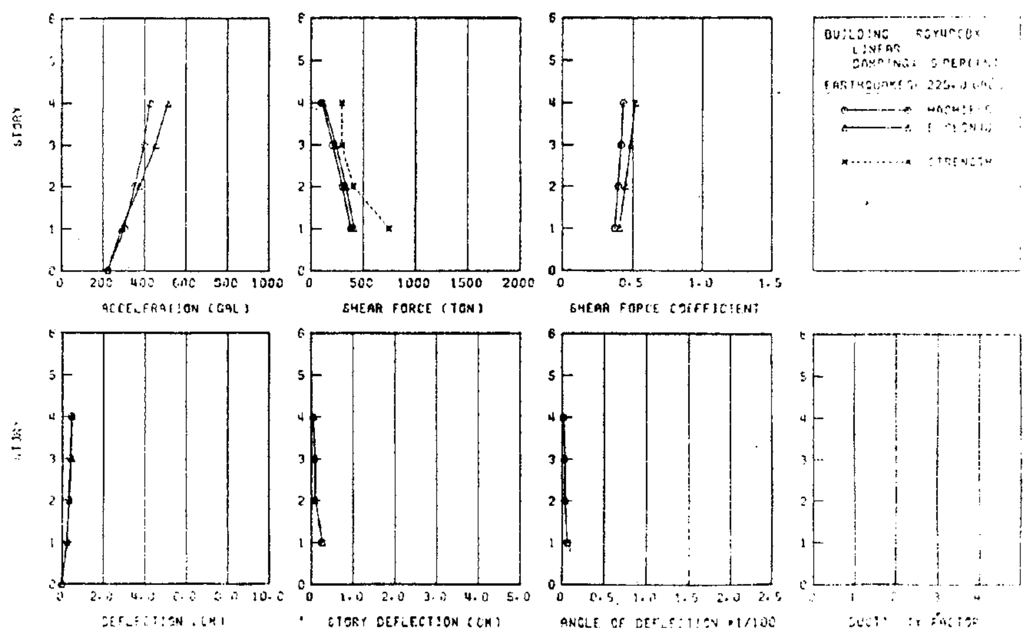
Before Retrofitting (B)

After Retrofitting (BR)

Fig.15b Exciting Functions of the Buildings B and BR

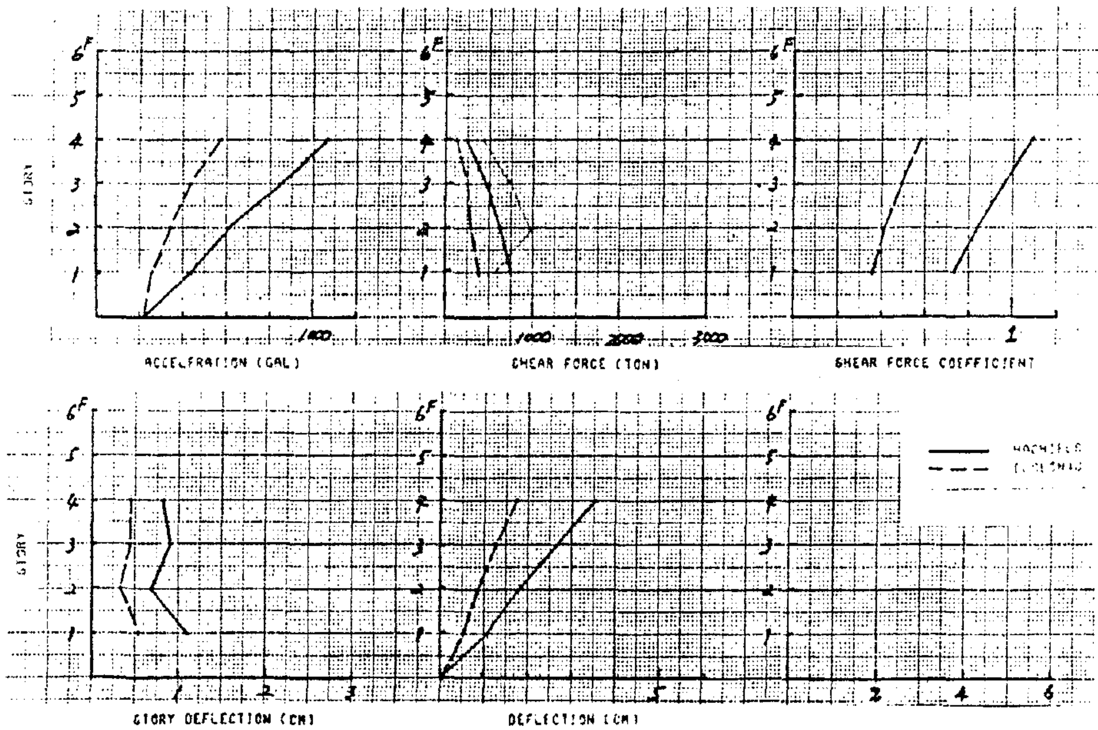


a. Building-A

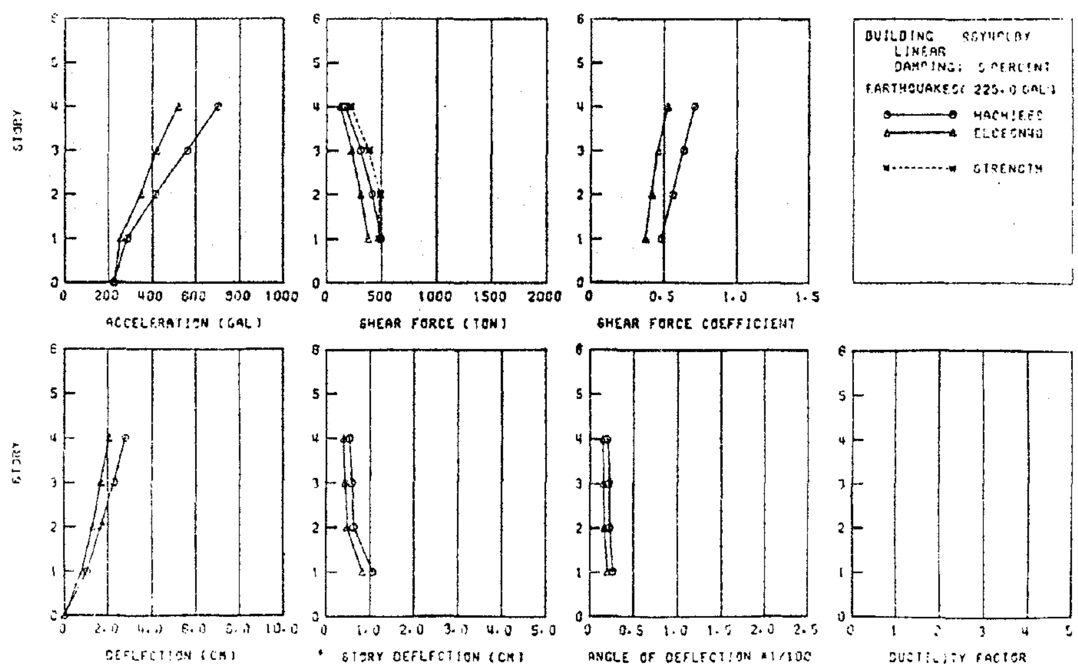


b. Building-A_R

Fig.16 The maximum Linear Responses in the Longitudinal Direction of the Buildings -A and -A_R Subjected to 225gal Ground Motions

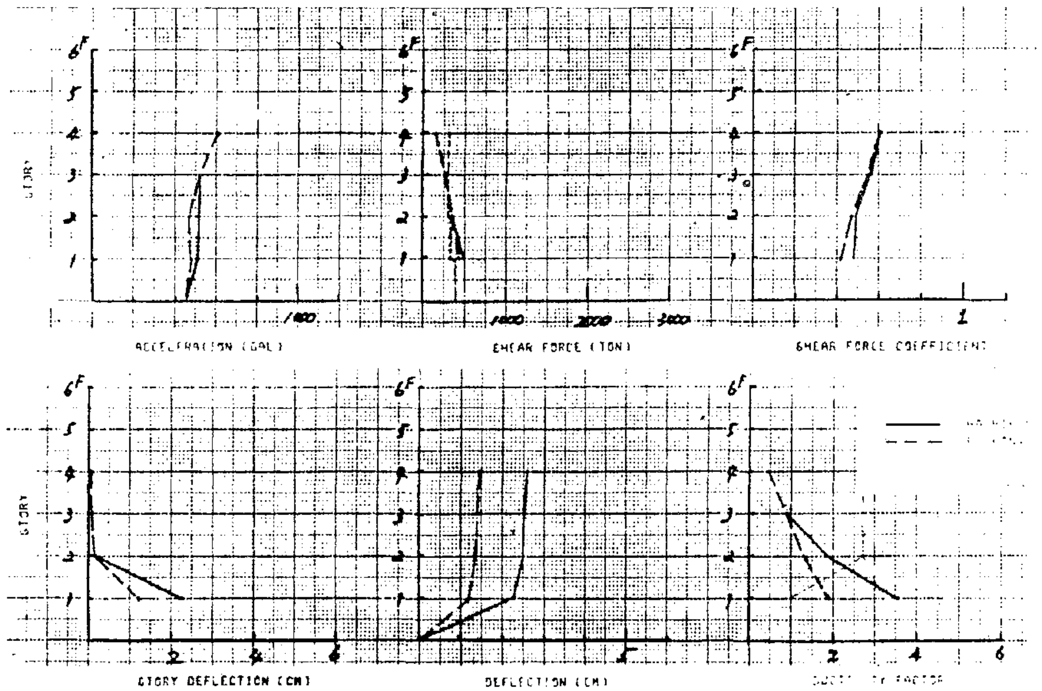


a. Building-A

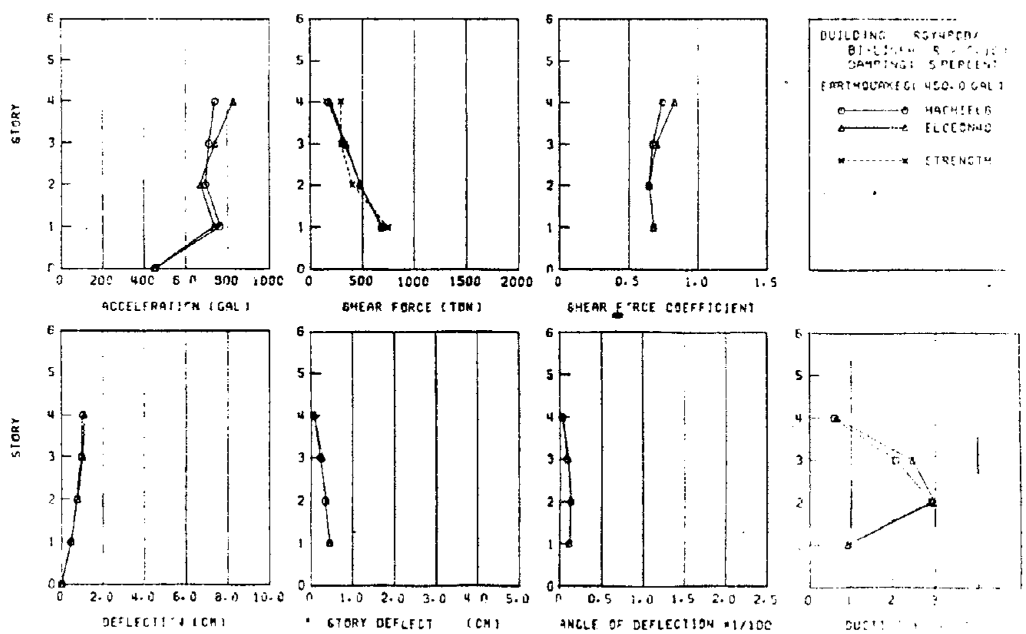


b. Building-A_R

Fig.17 The Maximum Linear Responses in the Transverse Direction of the Buildings -A and -A_R Subjected to 225gal Ground Motions

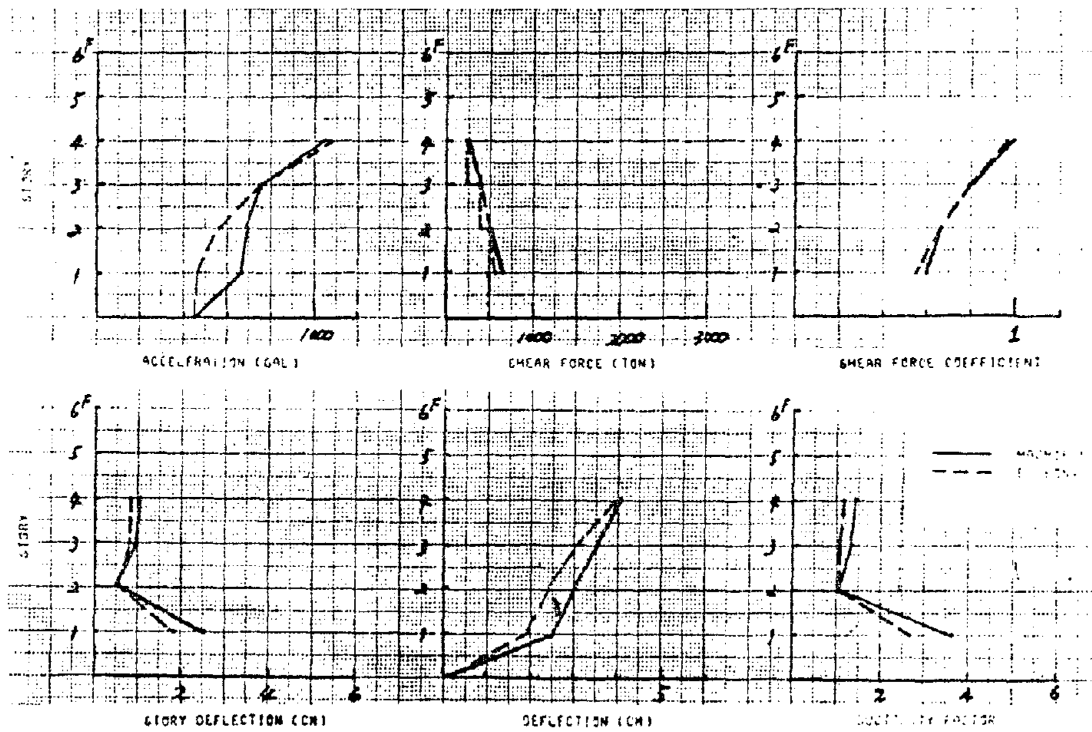


a. Building-A

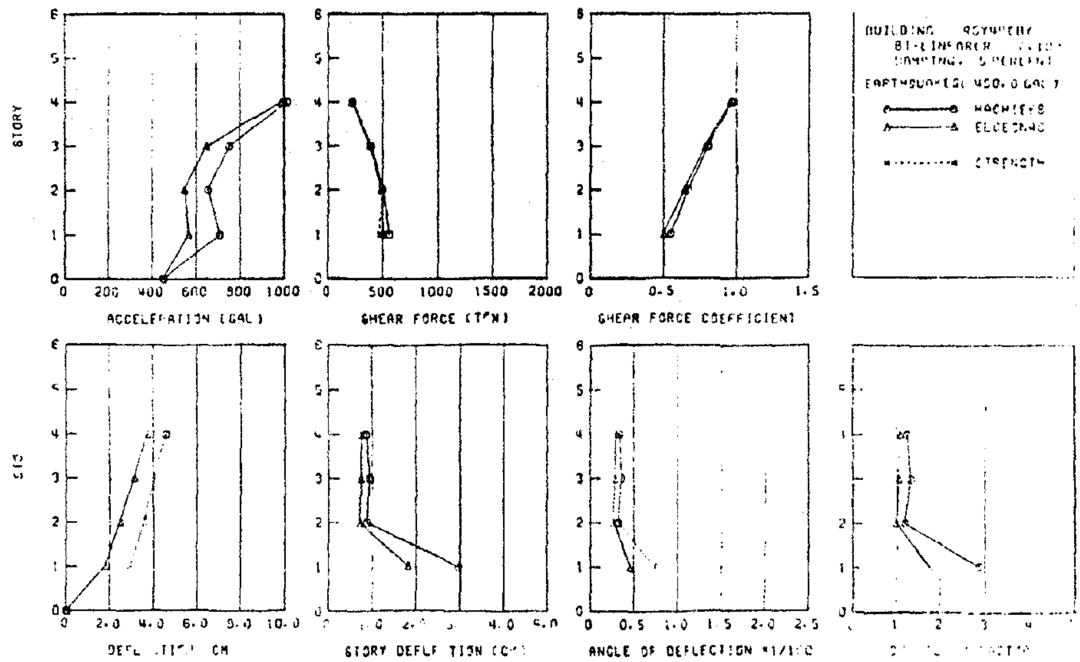


b. Building-A_R

Fig.18 The Maximum Bi-Linear Responses in the Longitudinal Direction of the Buildings -A and -A_R Sudjected to 450gal Ground Motions

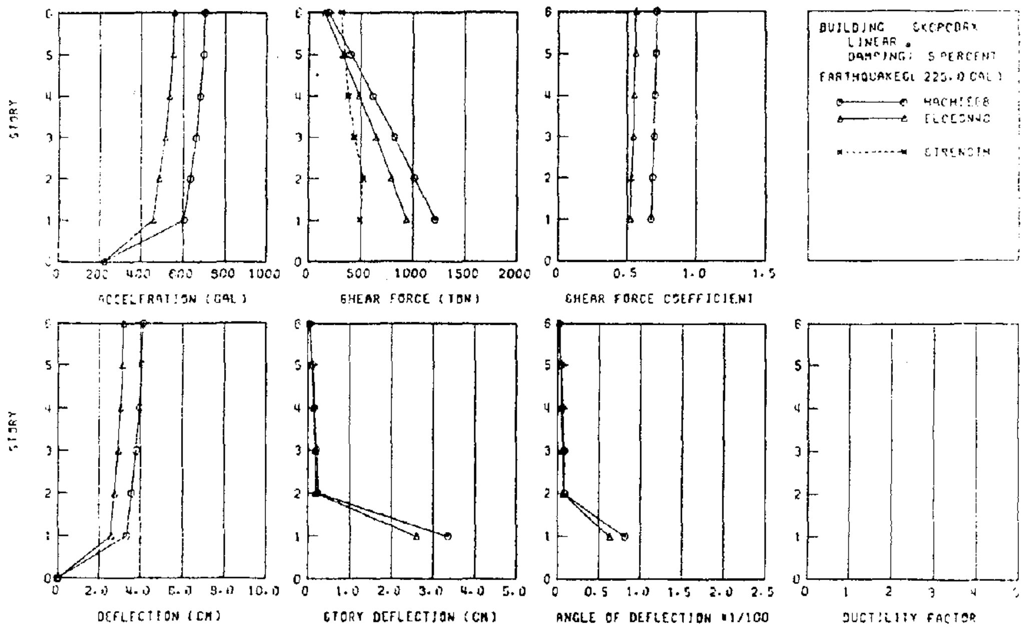


a. Building-A

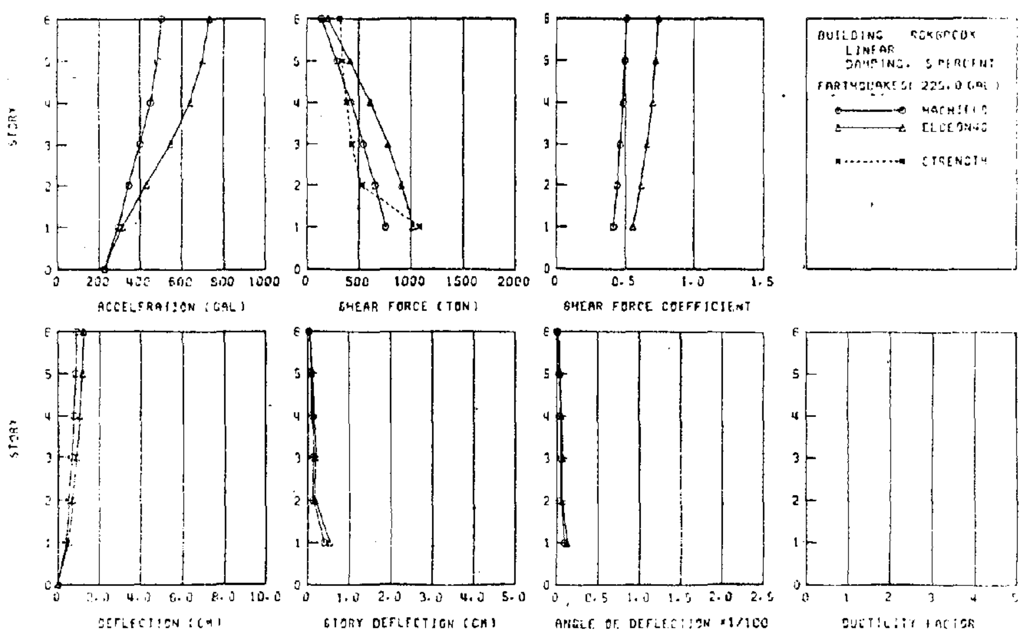


b. Building-A_R

Fig.19 The maximum Bi-Linear Responses in the Transverse Direction of the Buildings -A and -A_R Subjected to 450gal Ground motions

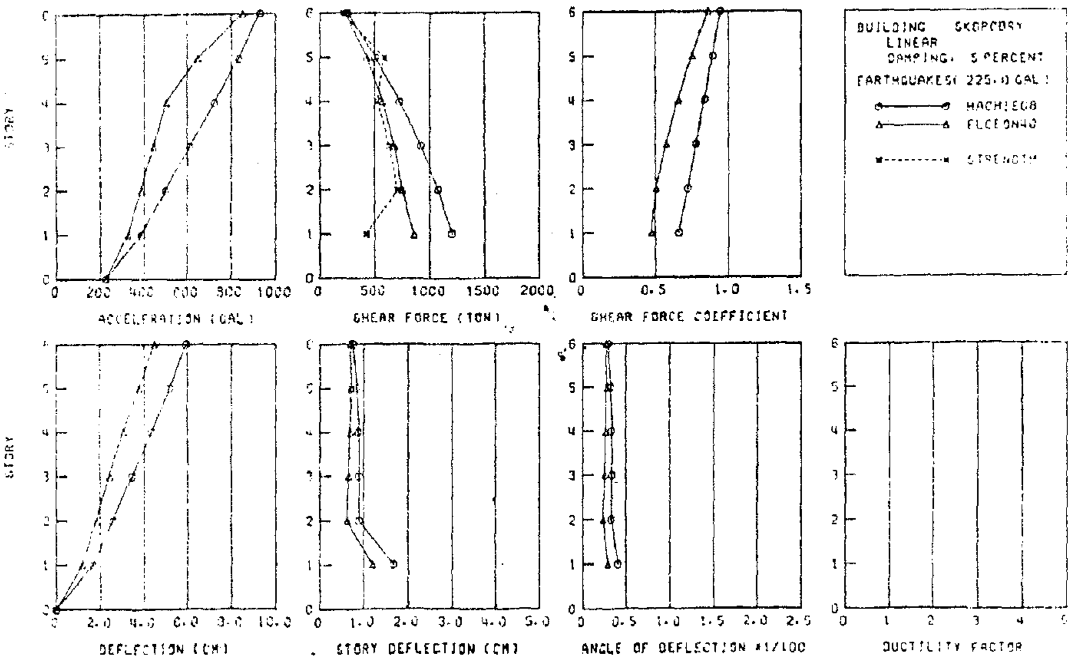


a. Building-B

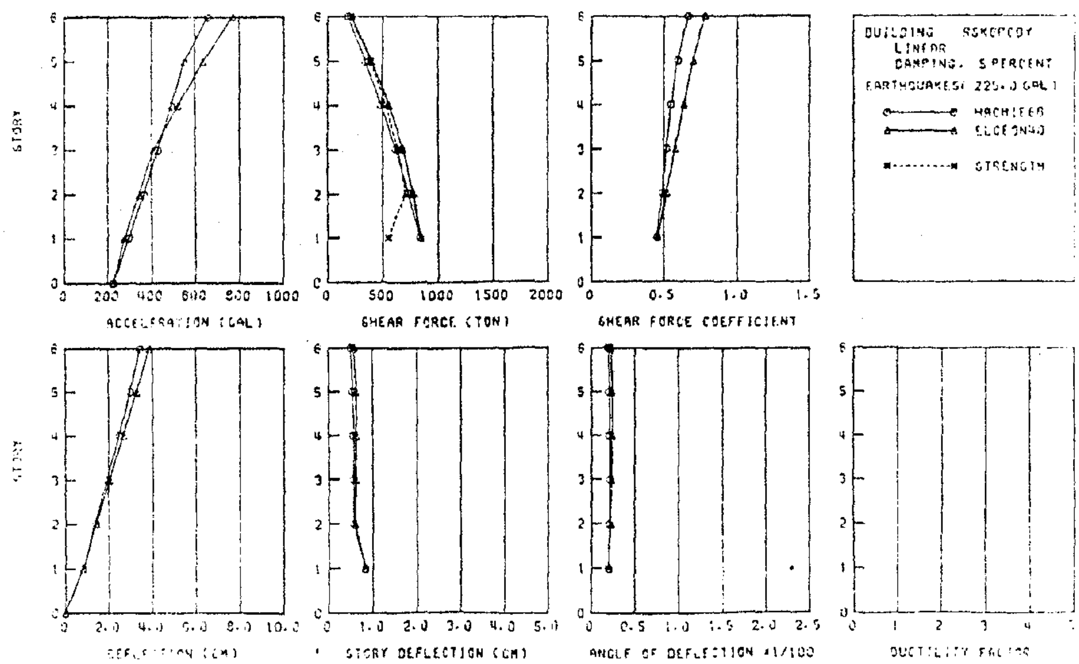


b. Building-B_R

Fig.20 The Maximum Linear Responses in the Longitudinal Direction of the Buildings -B and -B_R Subjected to 225gal Ground Motions

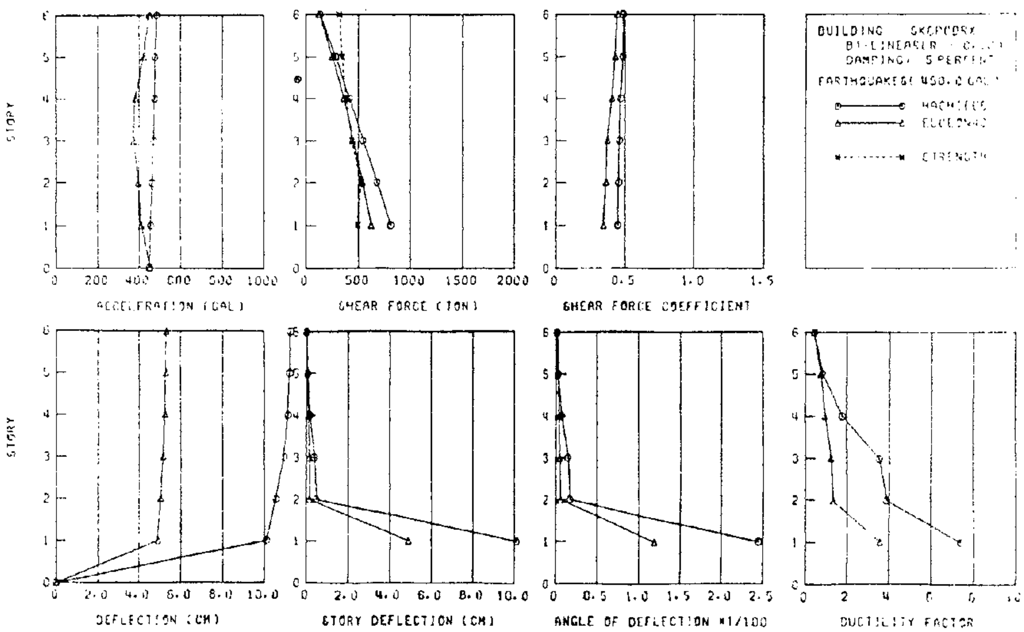


a. Building-B

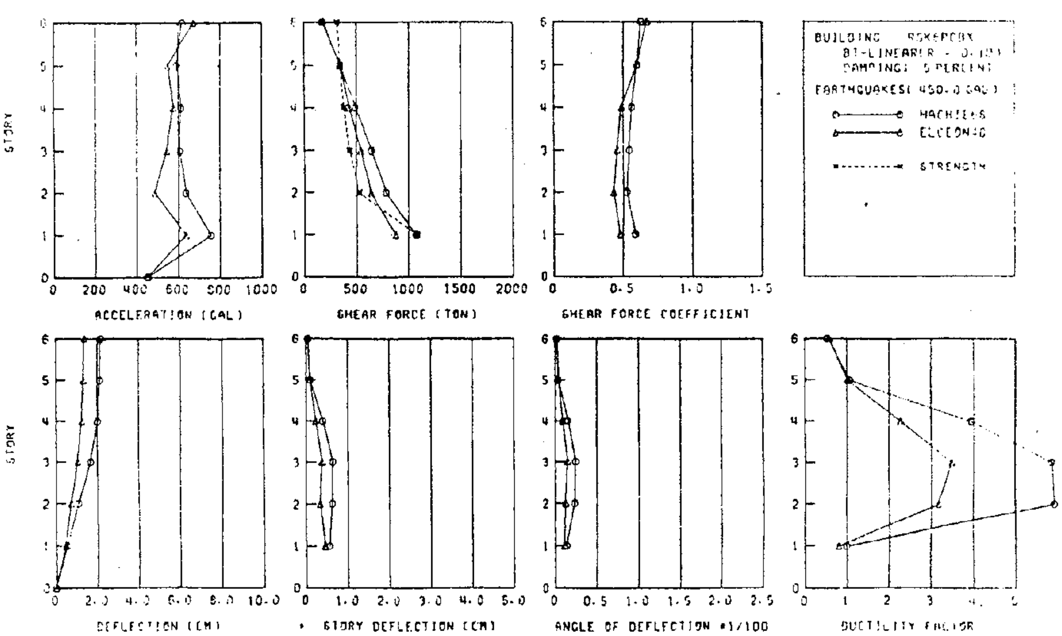


b. Building-B_R

Fig.21 The Maximum Linear Responses in the Transverse Direction of the Building -B and -B_R Subjected to 225gal Ground Motions

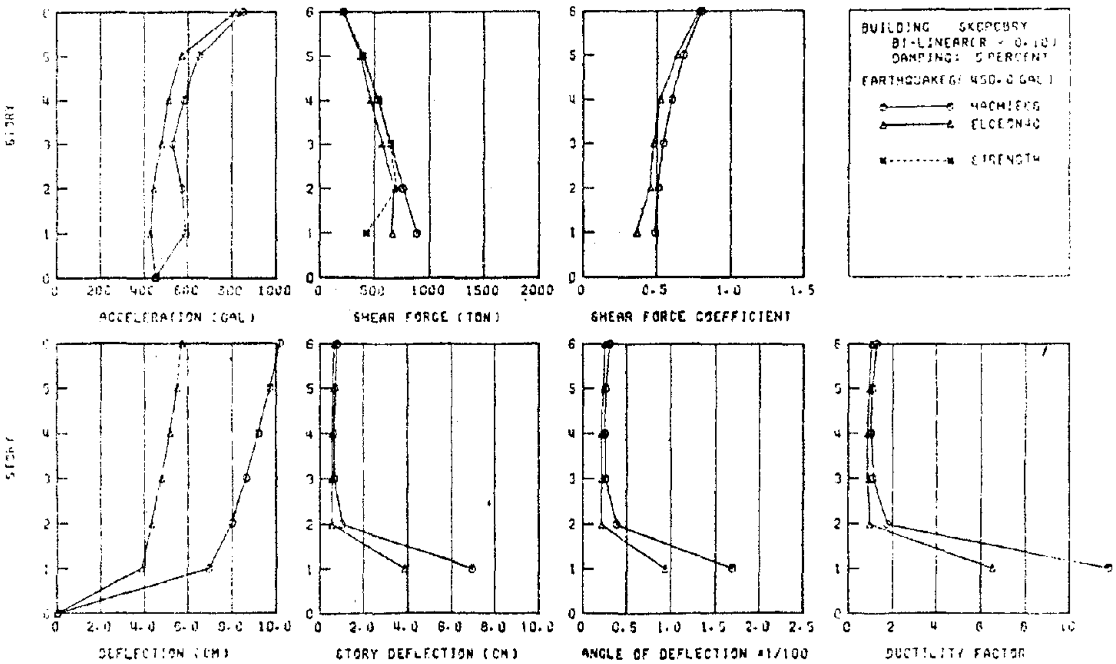


a. Building-B

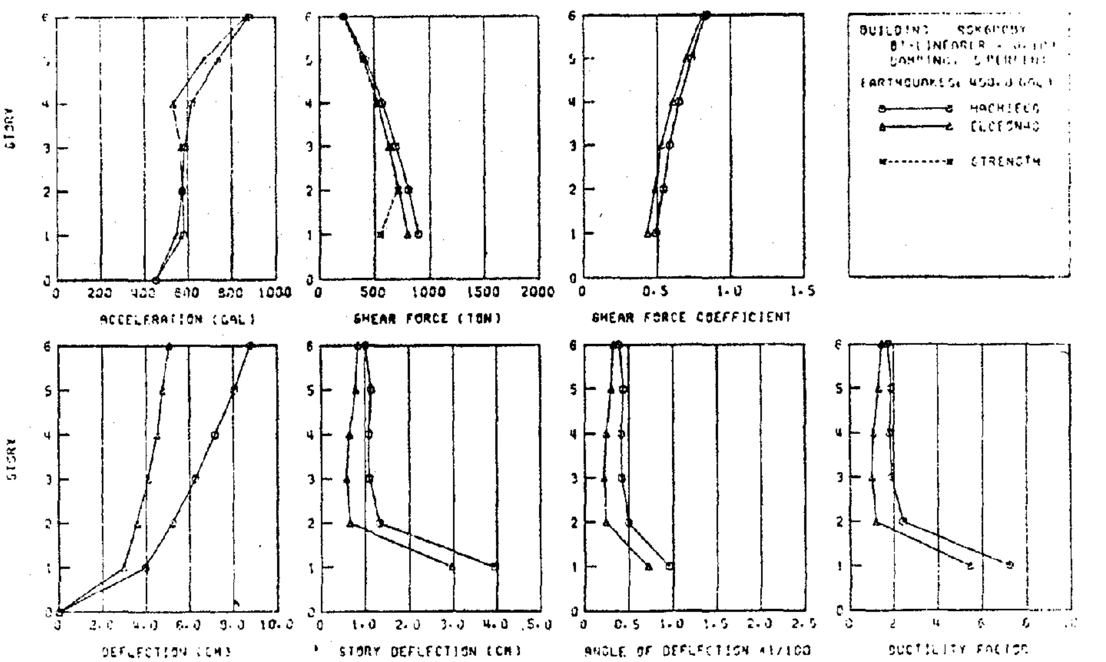


b. Building-B_R

Fig.22 The Maximum Bi-Linear Responses in the Longitudinal Direction of the Buildings -B and -B_R Subjected to 450gal Ground Motions



a. Building-B



b. Building-B_R

Fig.23 The Maximum Bi-Linear Responses in the Transverse Direction of the Buildings -B and -B_R Subjected to 450gal Ground Motions

Table 1 Natural Period

Buildings \ Modes \ Directions	Longitudinal (X) Direction			Transverse (Y) Direction		
	1st	2nd	3rd	1st	2nd	3rd
A	sec. 0.282	sec. 0.075	sec. 0.043	sec. 0.354	sec. 0.151	sec. 0.097
A_R	0.199	0.068	0.042	0.400	0.158	0.100
B	0.477	0.103	0.055	0.488	0.197	0.125
B_R	0.259	0.089	0.053	0.452	0.185	0.119

STRENGTHENING EXISTING CONCRETE AND MASONRY BUILDINGS
FOR SEISMIC RESISTANCE

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H.J. DEGENKOLB ASSOCIATES, ENGINEERS

For presentation at
Second Joint Meeting
United States/Japan Cooperative Research Program
in Earthquake Engineering on Repair and Retrofit
of Buildings and Lifelines
Tsukuba, Japan
May 18, 1981

SUMMARY

This paper outlines various methods currently being utilized for strengthening buildings for improved seismic resistance. It considers buildings damaged in earthquakes as well as buildings where voluntary considerations or mandatory regulations result in a strengthening program to improve potential seismic performance. Technical and non-technical aspects are discussed, as both are essential to any successful application of the goals of seismic strengthening.

INTRODUCTION

Strengthening existing buildings properly for improved seismic resistance is a difficult task involving many areas of study. The existing building must be thoroughly analyzed so the engineer is familiar with the strengths and weaknesses of the original lateral force resisting system. He must appreciate the functional usage of the building as well as its aesthetics since his strengthening scheme will impact on both. He must assess strengthening schemes utilizing different structural materials working in conjunction with the original structure. He must assess the impact of the strengthening scheme on the final structure to insure that new areas of dynamic weakness are not created. Finally, he must develop this scheme within the economical realities of construction considering the impact of the reconstruction on finishes, mechanical and electrical systems, etc. It is a complicated task to challenge the knowledgeable engineer.

There are three basic reasons why buildings are strengthened for improved seismic performance. First is to strengthen a building damaged in an earthquake and to improve its performance in future events. Secondly, many Building Codes or regulations, at least in the areas of high seismicity of the United States, require older buildings to be strengthened to the current code's seismic regulations when the usage of the building is changed to increase its occupancy or potential hazard. Finally, an increasing number of knowledgeable and concerned building owners are voluntarily strengthening selected buildings based on a concern for the safety of their employees and the protection of their financial investment.

This paper will outline some of the considerations appropriate to strengthening buildings for improved seismic performance. It will survey methods and techniques currently used in the United States to strengthen existing concrete and masonry buildings for added seismic resistance strength.

STRENGTHENING EARTHQUAKE DAMAGED STRUCTURES

Strengthening a damaged structure must be separated from repairing the damage, although the two are frequently considered together. Repairing the damage is the attempt to return the structure to its original strength. Strengthening the structure is judiciously increasing its strength and/or stiffness to improve the building's performance in future earthquakes.

The first step in strengthening any earthquake damaged structure is determining exactly how the structure performed. This requires a detailed inspection of the building and a listing of all damaged elements and members. It may be necessary to open concealed areas to permit a thorough investigation and insure that hidden damage does not remain undetermined.

The engineer must then analyze the structure and thoroughly understand why the damage occurred. He must satisfy himself of the force resistant paths in the building and why certain members failed or cracked while other members were essentially undamaged. He must determine if members failed due to shear, compression, tension, flexure, bar anchorage, etc. He must consider the effects of non-structural elements such as walls and parapets. This analysis is essential before any repairs can be designed.

Once the damage is documented and understood, the repair of individual members can be designed to return the original or desired strength to the member. Such repairs usually consist of epoxy injection, partial replacement or occasionally, complete replacement of the damaged member.

The engineer then needs to consider how to minimize such damage in the future. He may decide to strengthen selected members which failed and make them considerably stronger. He may decide to add shear walls to stiffen a frame structure. He may replace damaged non-structural walls with structural bracing walls.

The force level to be used in designing the strengthening scheme will generally be greater than that used in the original design. Frequently, the engineer will have to use his judgment in establishing the force level for the strengthening. He may select the current local code or another modern code which is acceptable to the local officials. He may use a site response analysis or strong motion data from local earthquakes to establish design levels for the reconstruction. Buildings with high degrees of torsion or strength or stiffness discontinuities should be given special consideration to overcome those potential areas of dynamic weakness, possibly even doubling up systems in the area of weakness. Th exact criteria selected should be appropriate for the damaged building and consistent with its uncalculated strength, inherent stability or lack of stability and its redundancy.

Strengthening the undamaged structure can frequently be a professional challenge. The structure has generally not been tested by a damaging earthquake so its actual performance is unknown. It most likely does not conform to the current building regulations. Frequently, a building in this category has a lateral force resisting system which is no longer permitted by code and considered acceptable, such as unreinforced brick walls, non-ductile conventional concrete frames or a discontinuous shear wall system.

Criteria for strengthening the undamaged building will generally be the current building code. This is particularly true of buildings with changing occupancies or usage where the strengthening is required by the Building Code. However, when the strengthening work is not required by ordinance or building regulation and is voluntary by the owner, other criteria may be more appropriate. For example, a building with a well proportioned shear wall system which is discontinuous in a single story can be strengthened in the discontinuous story only even though the shear strength of the original walls is somewhat less than current code.

When buildings are strengthened to mitigate selected structural deficiencies without bringing the building into full compliance with current code, several factors should be considered. First, reliance on structural systems proven inappropriate for seismic resistance must be eliminated, although these systems strengthened or braced may remain in the strengthened structure. An appropriate force level must be selected, either the original design force level or another suitable criteria. The proposed criteria and strengthening scheme should be reviewed in detail with the owner, as the strengthening to force levels less than current code is most likely saving the owner considerable money while hopefully preventing a collapse in a future earthquake but it may result in greater damage in that future quake than a structure strengthened to full current code value. The owner should share in this criteria decision, and understand that his strengthening investment is a form of insurance but not a guarantee to a damage-free building. The engineer should clearly explain the alternatives and his opinions of anticipated performance so the owner can intelligently share the decision with the professionals as well as the consequences. In this case of a voluntary strengthening not dictated by regulations, the local building official will usually be agreeable to the selected approach, although he should be contacted for concurrence.

METHODS OF STRENGTHENING STRUCTURES

Many methods are available to strengthen existing concrete or masonry buildings. What is appropriate for one building will be inappropriate for another. The methods selected must be consistent with aesthetics, building function, the original structure and its strength, ductility, stiffness and redundancy. Continued occupancy of the building during strengthening when required will have a major impact on the scheme selected. The vertical continuity of strengthening elements is extremely important and may require significant changes in functions on certain floors. The following paragraphs outline the usual methods for strengthening concrete or masonry buildings.

An apparently simple method is to add new cast-in-place reinforced concrete shear walls. When adding new walls, it is frequently desirable to locate them near but off the original column lines so good vertical continuity can be achieved at floor levels which is often difficult at column lines where heavy beams frequently exist and so new foundations can be added between existing footings to support the added weight of the walls. In taller structures, overturning may become a problem if the walls are too slender. It will also be necessary to engage existing columns for loads to counter uplift tendencies. Floor systems must be checked for diaphragm strength and chords or collectors may have to be added for proper stress transfers at each level. When all facets are considered, this method is not always as simple as it first appears.

The addition of new reinforced shear walls by shotcrete or gunite is a frequently used technique, especially for masonry buildings or frame buildings with masonry infill. This is particularly suitable for historic or elegant older structures where the aesthetic appearance of the building is to be maintained. The gunited wall should be well bonded to the original wall and contain through bolts or epoxied anchors to achieve a composite action of the old and new walls. The gunited section should contain sufficient reinforcement for the strengthening desired and holes must be cut or cored through the floor and possibly original columns to pass reinforcing steel. The gunite section must have sufficient strength and stiffness to span between floors for forces perpendicular to the wall which frequently requires chases to be cut in the original wall or the addition of pilasters, which might also serve to provide space for vertical trim reinforcement at window jambs. For such reinforcing systems, it is essential to carefully detail at large scale the exact location of reinforcement at all levels as holes must be cut in the structure to pass significant reinforcing steel which must be placeable in the proper location.

Another scheme which the author has recommended for several structures is the strengthening of existing frames to develop reasonable shear wall systems. This approach is particularly suited to buildings with perimeter framing systems of slender columns and deep, stiff spandrel beams which were a popular form of construction in California in the 1950s and 1960s. Experience has shown that buildings of this type with strong beams and weak columns lead to column failures with building instability and potentially hazardous conditions. Strengthening can be similar to Figure 1, with the columns strengthened to a point where they are compatible with the beams and will perform satisfactorily as a shear wall system. Holes must be cut in the floor system to pass new vertical reinforcement and great care must be taken to enhance the integral working of the original and new concrete. The greatest concern with this type of strengthening is insuring that the old and new concrete will behave in a monolithic condition.

Strengthening concrete or masonry buildings with concrete is not always the proper solution. Frequently, the most appropriate solution is with new structural steel framing, most likely a braced steel frame. A major advantage of using structural steel systems is the negligible addition of weight to the structure, thus preventing a significant increase of mass and resulting lateral forces as well as minimizing potential foundation strengthening for increased gravity loads which can prove to be very expensive. Generally, the steel bracing will be in the form of a braced steel frame along a portion or all of the building's perimeter. The pattern of the bracing is usually selected to be compatible with the window and doorway locations as well as providing a pleasing aesthetic appearance when exposed. Braced frames with eccentric brace locations, as has been shown by experiment to provide excellent energy absorption for earthquake resistance, can also be used to provide more adaptability of the bracing scheme to suit the window and doorway pattern of the building. Figure 2 illustrates several possible bracing schemes.

Moment resisting steel frames can also be utilized to brace concrete buildings where the diagonal members of the braced frames are found objectional to the functional or aesthetic considerations of the building. The moment resisting frames can be simple bents placed within the concrete framing with appropriate details at the floors to transfer shear and overturning forces. The structural steel weight required for this type of bracing can be significantly higher than for braced frame systems.

One consideration important to the concrete or masonry structures being strengthened with a structural steel system is the relative rigidities of the original concrete or masonry structure and the new steel bracing. Generally, the original concrete or masonry structure will be many times more rigid yet we are relying on the new, relatively flexible steel system for strength, stability and ductility. In an earthquake, we must expect cracking in the original concrete or masonry structure, and after sufficient cracking has occurred, the new steel system will have comparable stiffness and be effective. Designs of this type are completely valid as crack widths can generally be accepted in the original structure. If the original structure contains totally non-ductile materials like unreinforced masonry, additional measures may be necessary to maintain integrity during this cracking phase.

For all steel bracing schemes, the most important design considerations are the transfer of forces between the steel bracing and the concrete structure. This involves both the horizontal input at each floor level to the steel bracing as well as transferring vertical uplifts back into the concrete columns to control overturning tendencies of the lightly loaded steel. Chords and collectors for the horizontal diaphragm often must be added. It is the author's opinion that the steel bracing should provide a complete system, consisting of horizontal steel members at the floor level to collect the seismic forces, continuous steel members (probably adjacent to existing columns) to resist overturning forces, and diagonal members in between to resist the shears. Connections between the new steel and original concrete require special consideration - epoxy bolts, conservatively designed expansion anchors in shear, or new concrete encasing the steel and well bonded to the original concrete.

Fireproofing of the added steel bracing must be considered and reviewed with the local Building Official or Fire Marshal. As part of a fireproofed structure, the steel bracing would generally require fireproofing consisting of approved materials. However, the fireproofing of such bracing is really beyond the state of the art of structural fireproofing and the likelihood of a fire being in progress at the time of an earthquake is extremely remote. Since the steel bracing is added for seismic resistance alone and not vertical load stability, a case can be made that the steel bracing need not be fireproofed. However, if the earthquake starts a fire in the structure, the steel bracing should maintain its integrity so it can resist aftershocks. This appears to be an area of needed research with fire protection engineers working in concert with structural engineers to reach a reasonable solution. Most likely, steel bracing on the building's exterior surface has an inherent fire resistance while interior bracing solutions may require appropriate fire protection.

Concrete frame structures can and have been strengthened for seismic forces by strengthening the concrete frames. Depending on the proportions of the frames, columns have been strengthened by a new jacket of concrete containing added vertical reinforcement as well as closely spaced ties. Beams can be strengthened with new confined beams with continuous reinforcement and closely spaced ties each side of the original beam or a new beam replacing one that has been removed. This type of system has seldom, if ever, been used in the United States since the resulting structure should comply with the "ductile concrete" frame provisions of American codes. However, this approach has been used in various parts of the seismically active world and should provide reasonable protection to occupants with appropriate details and sound engineering judgment accompanying the engineering solution.

Another approach to strengthening buildings for seismic resistance, completely divorced from the previous schemes,is the addition of external buttress structures. These buttress structures must be located adjacent to the original structure and generally contain massive shear wall systems around their perimeter. They must be thoroughly connected to the original structure. Although this may be a more expensive system, it provides additional area to the building which may offset the added cost. Figure 3 illustrates this approach to this strengthening problem.

GENERAL CONSIDERATIONS

Regardless of the strengthening scheme selected to satisfy the structural requirements, there are other related factors that must be considered in the process. This section of the paper attempts to outline those considerations.

Any building must be a pleasant one to occupy if it is to be a successful building. The occupants of the building must feel confidence in their surroundings and be happy in their environment. When a building is damaged in an earthquake or is declared potentially hazardous by a structural engineer, the occupants become apprehensive and lose confidence in their surroundings. Thus, an important part of any seismic strengthening procedure is a public relations effort with the building's occupants to establish a confidence in the strengthened building. If the occupants are not satisfied, all technical efforts are lost and the project becomes a failure.

Aesthetics is another important consideration. If the strengthened building looks like a bunker on some military front, it will never gain acceptance by the using public and the end product will become an economic failure. As structural engineers involved in strengthening buildings, we must consider the human aspects, work with our architect colleagues, and create pleasant environments for future generations.

The structural strengthening must also consider the entire structure and potential consequences. It is a waste of money and effort to strengthen a weak link in a structure only to transfer potential failure to the next weakest link. The following two examples illustrate this situation.

The first example involves the Colegio Teresiano on the outskirts of Managua, Nicaragua. The building is a three-story concrete frame school building of a long rectangular plan, similar to schools built throughout the world. A small earthquake of magnitude 4.6 in 1968 was centered quite close to the building and caused cracking and structural distress to the columns in the first story. The building was repaired by adding a stiffened concrete wall element in the first story between classroom doors and extending up to the second floor balcony rail height. This new wall element can be seen in Figure 4. The destructive Managua earthquake of December 23, 1972, caused considerable damage to this building, but only in the second and third floors, where considerable column damage resulted. Figure 4 was taken after this second earthquake. The new wall elements in the first floor prevented damage in that floor, but permitted the earthquake forces and motion to travel upward, causing the observed damage. The repairs had not considered the effect on the remainder of the structure. Had these or stronger walls extended to the roof, much of this damage might have been prevented.

A second example shows a three-story classroom building at the Agricultural University in the La Molina area of Lima, Peru. There are four identical buildings of concrete construction. The first story was originally framed without structural walls and only columns for support and bracing. Considerable wall panels and masonry partitions were present in the upper two stories. A magnitude 7.5 earthquake on October 17, 1966, caused significant damage to the first story columns, so concrete shear panels were introduced to stiffen and brace this first story. A second earthquake of magnitude 7.6 affected these structures on October 3, 1974. Figure 5 shows the end of one of these buildings after that earthquake. There was little damage in the first story due to the previous strengthening, but that increased stiffness caused considerable damage in the upper two floors which had not been strengthened after the 1966 earthquake.

Seismic strengthening schemes must consider all their consequences, and provide an acceptable solution without merely transferring potential distress to the next weakest link of the building. This is a most important consideration for the structural engineer responsible for the redesign. He must fully appreciate the consequences of his redesign and competently satisfy the purpose of the strengthening effort.

One final consideration in any strengthening scheme which is of particular importance to the owner is the length of time required to complete the strengthening as well as the condition of the building during the process. In this day and age of high interest rates, shortage of usable space and environmental considerations, it may be essential to the owner that full or partial occupancy be maintained during the reconstruction for seismic strengthening. Such considerations become a primary factor in any solution scheme, generally requiring work on the building's perimeter, probably with premium wages being paid for noisy work at non-peak hours. These factors will frequently dictate the strengthening scheme, and will require full consideration by the stuctural engineer early in his efforts.

CONCLUSIONS

This paper has attempted to survey the methods of strengthening of existing buildings for seismic considerations. The methods are applicable to buildings damaged in earthquakes as well as buildings being strengthened to mitigate potential hazards discovered by routine review. Non-structural factors, such as the need to partially occupy the building during the reconstruction, must be given prime consideration and may dictate the actual strengthening scheme selected.

Various seismic strengthening schemes are outlined in this paper. These include new concrete cast-in-place shear walls, new shear walls by the shotcrete or gunite process, conversion of existing non-ductile frames to an acceptable shear wall system and strengthening solutions incorporating structural steel bracing, either by braced or moment resisting frames. Buttress additions to the building are also considered.

Whatever strengthening system is selected for technical reasons, it must be compatible with aesthetics, the building's environment and the functional requirements of the building. In a technical sense, it must not provide strength only in isolated areas while transferring potential severe damage to other weaknesses in the structural fabric. Seismic strengthening to existing buildings is a complicated, multi-disciplinary task involving the maximum dedication and attention of the design profession.

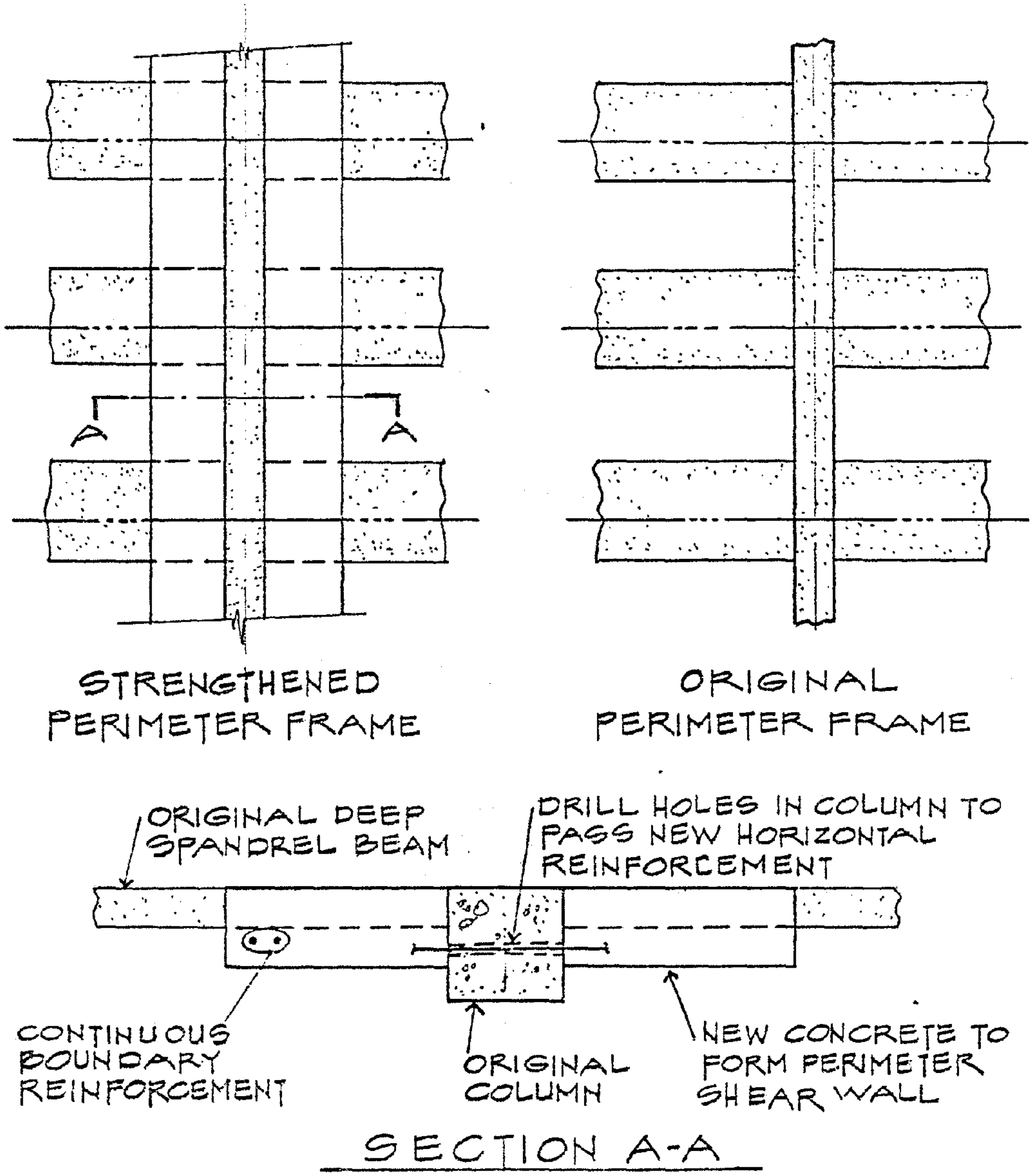
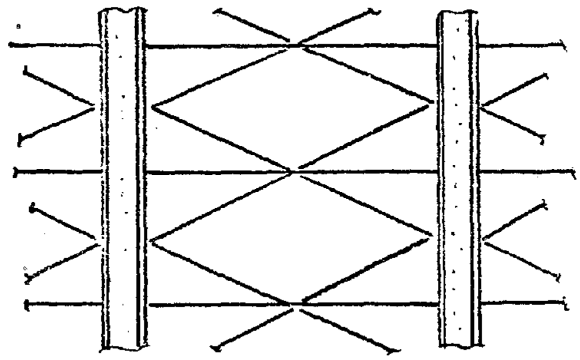
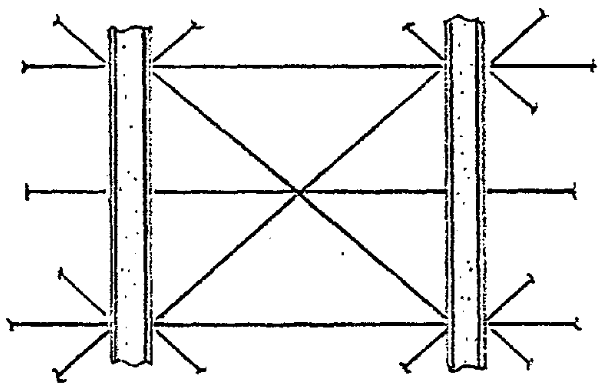


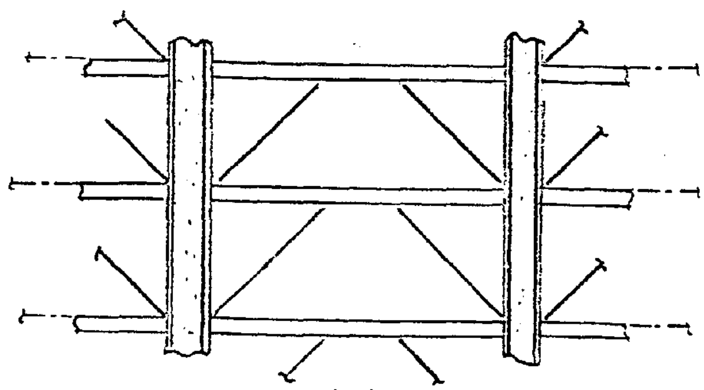
FIGURE NO.1. PERIMETER STRENGTHENING OF FRAME BUILDINGS. BOND AND ANCHORAGE OF NEW CONCRETE TO ORIGINAL CONSTRUCTION IS ESSENTIAL IN THIS TYPE OF RECONSTRUCTION.



(a)

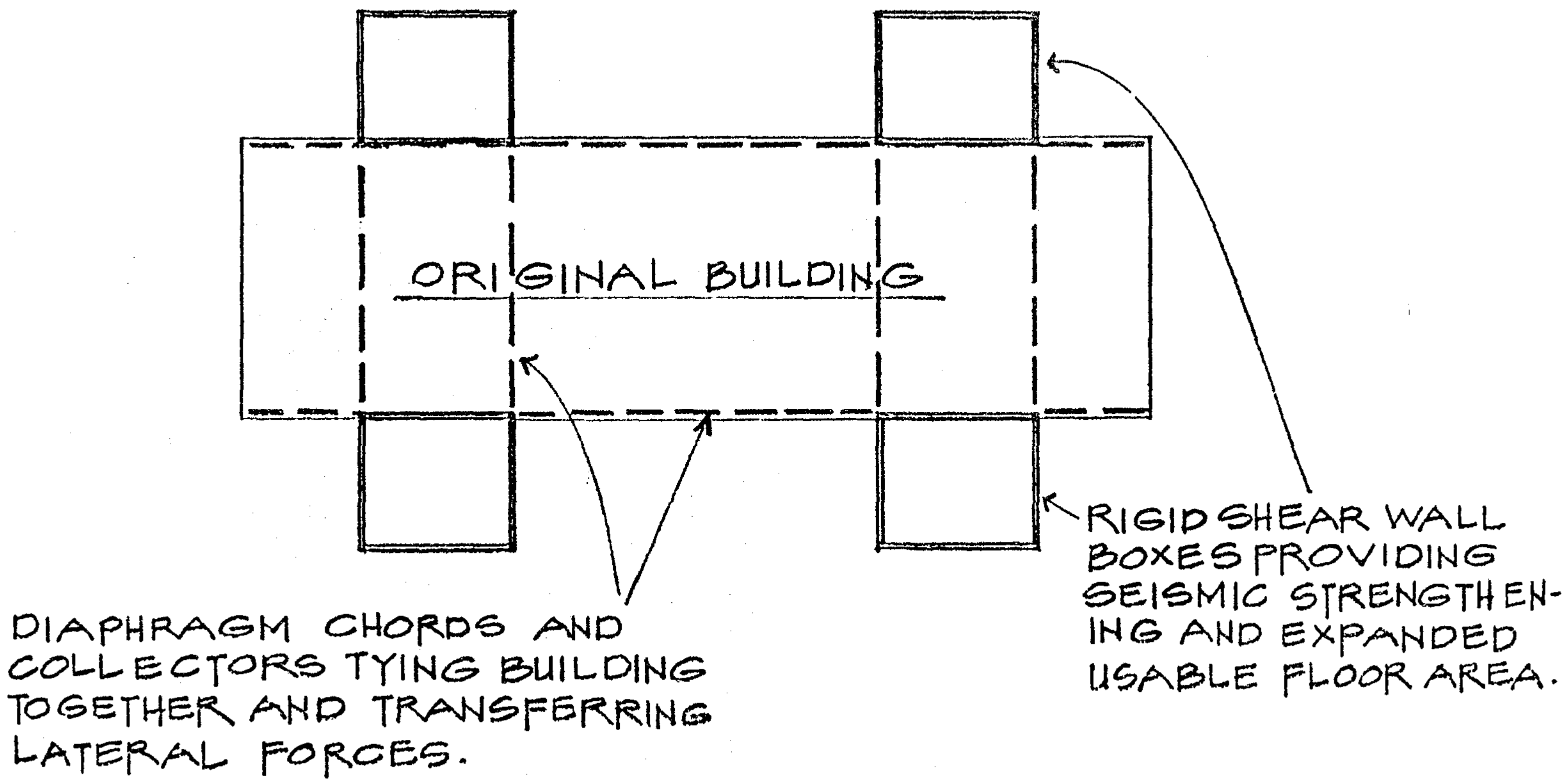


(b)



(c)
ECCENTRIC BRACING

FIG. NO. 2, EXAMPLES OF STEEL BRACING SCHEME FOR STRENGTHENING CONCRETE FRAMED BUILDINGS.



PLAN

FIGURE NO. 3

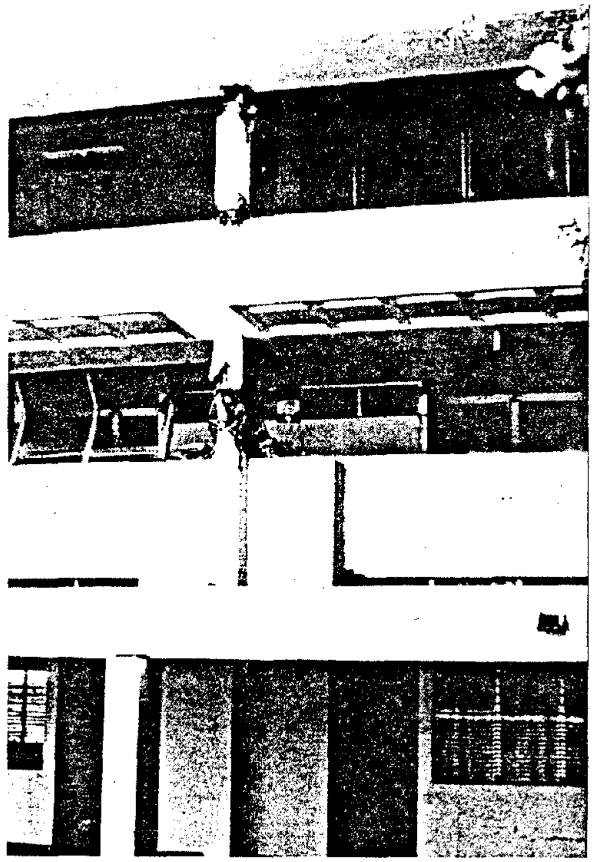


Figure 4. Colegio Teresiano in Managua, Nicaragua, after 1972 earthquake. First story stiffening wall, which can be seen projecting outward from second floor beam, prevented first story damage but increased upper story damage.

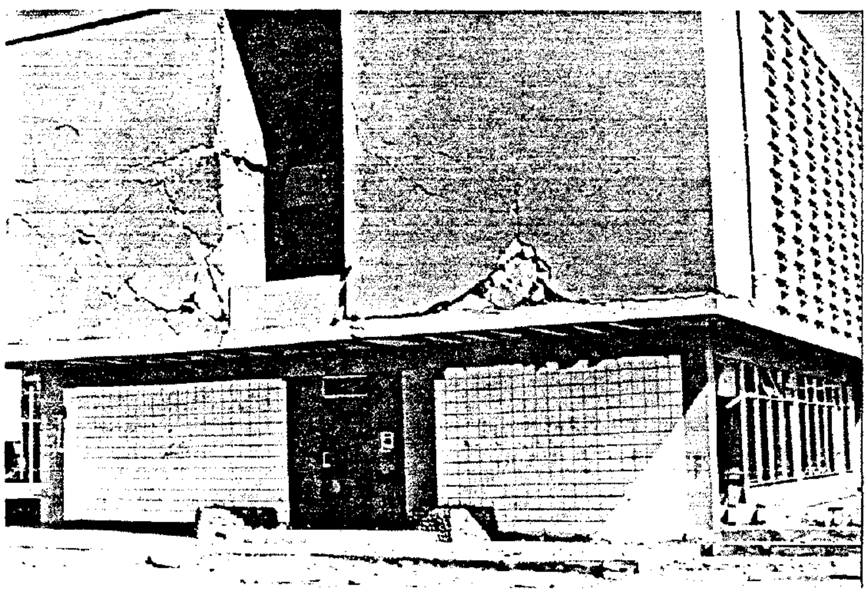


Figure 5. Classroom building at Agricultural University after 1974 earthquake. Stiffened first story had little damage due to added concrete wall panels, but upper stories had increased damage in this earthquake.

EARTHQUAKE DAMAGE AT IZUMI HIGH SCHOOL IN 1978 MIYAGI-KEN-OKI EARTHQUAKE AND METHODS OF REPAIR AND STRENGTHENING

M. Yokoyama[I] and H. Imai[II]

SUMMARY

The paper describes earthquake damage at Izumi High School and methods of repair and strengthening. Damage was concentrated in the ridge direction which did not have shear walls, and especially at the first floor where columns were short due to the existence of spandrel walls the columns suffered severe shear failure. Fairly damaged columns were repaired by grouting epoxy resin while the most severely damaged columns were demolished and new concrete was placed. Shear walls were newly provided in open frames to strengthen the building against future earthquakes.

§1. INTRODUCTION

The Miyagi-ken-oki Earthquake which occurred on June 12, 1978 caused damage to a large number of buildings centered at the city of Sendai, Japan. Reinforced concrete school buildings suffered fairly heavy damage. The Izumi High School building was the only one severely damaged among buildings designed according to the current standards of Japan. The school was situated on hard ground, but since there were no shear walls in the ridge direction of the structure, columns sustained severe shear failure in the ridge direction.

Repairs on the building consisted mostly of work done on damaged columns. In order to systematically carry out repair work, all columns were classified according to five levels of damage with methods of repair using epoxy resin proposed according to degree of damage. Columns damaged to the greatest extent were demolished and new concrete was placed. Further, in order to be prepared against future earthquakes, shear walls were newly provided. The repair work was completed in the spring of 1979 and the building is presently being used in the same manner as before the earthquake.

§2. OUTLINE OF BUILDING

This high school building, as shown in Figs. 1, 2 and 3, consists of three 3-storied blocks, A, B and C, and two 2-storied connecting corridors, and these are separated structurally by expansion joints at four locations. The plan of each block is typical of Japanese school buildings being in a straight line with ordinary classrooms at the south side, a corridor on the north side, and special large classrooms at both ends of the block. Because of this, there are many reinforced concrete walls between classrooms in the span direction, but no shear walls at all in the ridge direction.

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The structural design of the building was in accordance with current calculation standards for reinforced concrete by the Architectural Institute of Japan. The seismic coefficient was $k = 0.18$, strength of ordinary concrete was $F_c = 180\ kg/cm^2$, and reinforcement consisted mostly of deformed bars. The building was constructed during August 1972 to March 1975 adding blocks in the order of A, B and C.

§3. RESULTS OF DAMAGE SURVEY

The features of the damage were as described below.

(1) Damage to columns in the ridge direction was severe in all blocks whereas damage in the span direction was slight.
(2) Shear failure of north-side first-story columns was severe, followed by failure of south-side first-story columns, while interior columns were almost all undamaged.
(3) Damage to beams and spandrel walls was slight in both directions.
(4) Damage to expansion joints was also slight.
(5) On the whole, damage to Block C was heavier than to A and B.
(6) No damage was recognized at the foundation of the building and surroundings.

Since structural damage was mostly concentrated at columns, all of the columns were diagnosed in order to carry out repair work systematically, and damage was classified according to the five levels indicated in Table 1. As shown in Fig. 4, the damage was worst at the north-side first story of Block C. Examples of diagnosed damage are shown in Photo. 1 and 2.

The results of tests on reinforcing bars and concrete cores (diameter 100 mm, height 200 mm) taken from the building during repair work are given in Tables 2 and 3. The test results for reinforcing bars are normal, but the compressive strengths of concrete cores from Block C are low agreeing well with the features of earthquake damage.

§4. REPAIR AND STRENGTHENING METHODS

The columns of the various damage levels were repaired by the methods indicated below.

Damage Level 0 (No Damage)
(1) Surface finish is applied.

Damage Level 1 (Slight Bending Cracks, See Fig. 5)
(1) Portions of spalled mortar are fixed with lock pins.
(2) Cracked portions are taped together after which sealant is applied.
(3) The tape is removed and epoxy resin is injected successively from the bottom.
(4) Surface finish is applied.

Damage Level 2 (Small Shear Cracks or Large Bending Cracks, See Fig. 6)
(1) Mortar at spalled portions is chipped off.
(2) After providing injection holes cracked portions are sealed.

(3) Epoxy resin is injected successively from the bottom.
(4) The base course is finished with epoxy mortar and surface finish is applied.

Damage Level 3 (Severe Shear Cracks, See Fig. 7)
(1) Mortar and cover concrete are chipped off.
(2) After providing injection holes the entire surface is sealed with epoxy mortar.
(3) Epoxy resin is injected successively from the bottom.
(4) Epoxy resin for making construction joints is applied.
(5) Formwork is erected and non-shrink mortar is pumped in.
(6) Surface finish is applied.

Damage Level 4 (Severe Shear Failure, See Fig. 8)
(1) Window sashes and spandrel walls are removed.
(2) Temporary supports are installed around columns to hold up upper floors.
(3) Column concrete is demolished and part of the main reinforcement is cut.
(4) After replacing reinforcement, high-early-strength concrete is placed.
(5) Non-shrink mortar is pumped into the gaps at the tops.
(6) Spandrel walls are placed and window sashes are installed.
(7) The base course is finished with mortar and surface finish is applied.

After repairing damage to columns, in order to increase horizontal strength in the ridge direction, shear walls were newly provided mainly in the north-side planes as shown in Fig. 9 and according to the principles below.

(1) Wall thickness is increased in order that ultimate horizontal strength will not be determined by shear failure of shear walls.
(2) The quantity of wall reinforcement is increased in order that widths of shear cracks will not become large.
(3) Reinforcing bars are anchored in columns as shear connectors in aiming to make columns and walls integral.

Shear walls were added as described below based on the above principles (see Fig. 10).

(1) Substrate mortar of columns to be adjacent to walls was chipped off.
(2) Deformed bars were fixed with adhesive anchor at columns connecting to both sides of walls and beams above and below the walls.
(3) Wall reinforcement was placed followed by placement of concrete.
(4) Non-shrink mortar was pumped into the gaps between the walls and beams above.
(5) After finishing the base course of mortar the surface finish was applied.

§5. HORIZONTAL STRENGTH POSSESSED BY BUILDING BEFORE DAMAGE

Since the structure in the ridge direction was a pure frame with no shear walls, it was assumed that inflection points would be produced at mid-portions of beams when subjected to horizontal force, and as shown in Fig. 11, the columns were separated at the middles of beams to determine the horizontal strengths possessed. In calculations of strengths of beams against flexure and shear it was considered that spandrel walls would be structurally effective. However, in case of a doorway to a balcony on the south side, it was considered there was no spandrel wall for half of a span. The strengths of materials were taken to be the values in the results of investigations of samples shown in Tables 2 and 3.

According to the results of analyses, north-side columns yield in shear failure at the first story because there are spandrel walls. The horizontal strengths possessed at the first story obtained by simple summation of ultimate strengths of all columns were 0.56x, 0.57x and 0.48x the weight of the building at Block A, Block B and Block C, respectively, as shown in Table 4. Of these, north-side columns yielding in the form of shear failure comprised approximately one half. That the strength of Block C was relatively low was due to concrete strength being low.

Because there were tall spandrel walls at north-side columns, the stiffness of north-side columns against horizontal force was greater than for south-side and interior columns, and the maximum strength was reached at a small deformation angle. In case it is assumed that the ratios of stiffnesses of the various planes against horizontal forces until the north-side planes attain the above maximum strengths are the same as under elastic conditions, the north-side columns would reach maximum strength at approximately 0.80x of the horizontal strengths possessed previously described.

§6. HORIZONTAL STRENGTH POSSESSED AFTER REPAIR AND STRENGTHENING

It was assumed that inflection points would be produced against horizontal forces at the middles of boundary beams on both sides of 3-storied and 1-storied shear walls, and analysis models were made separating them at these points. With the analysis models, shear walls were considered to be the same as columns as shown in Fig. 12, and taking into consideration lifting up of the foundation, flexural strength was provided for support under the walls. Columns repaired through injection of epoxy resin were calculated as being completely recovered from damage.

According to the results of analyses, the failure mechanism of a structure containing a shear wall is that of yielding of boundary beams with maximum strength ultimately reached with lifting up of the foundation and the shear wall not failing in shear.

The horizontal strength possessed determined by simple summation of the ultimate strengths of shear walls and columns is approximately 0.75x the weight of the building at the first story as shown in Table 4. Of this, approximately 60% is that of the north-side structural plane containing shear walls.

§7. CONCLUSIONS

The methods of repair and strengthening of the Izumi High School building which suffered heavy earthquake damage have been described. Repair methods using epoxy resin in accordance with the degree of column damage were proposed and actually practised. Shear walls were newly provided in preparation against future earthquakes.

According to the results of analyses, the horizontal strength possessed at the first story before earthquake damage was approximately 0.5x the weight of the building and columns at the north side of large stiffness against horizontal force would fail in shear under smaller earthquake force. The horizontal strength possessed at the first story after repair and strengthening was approximately 0.75x of the weight of the building. The failure mechanism is that of bending yield of boundary beams and shear walls do not show shear failure. Consequently, strength reduction after reaching maximum strength would be small.

The Izumi High School building sustained severe damage in the Miyagi-ken-oki Earthquake, but it is believed it has been repaired and strengthened to be adequately earthquake-resistant.

ACKNOWLEDGEMENTS

The authors sincerely thank Professor Toshio Shiga, Tohoku University, and Professor Hiroyuki Aoyama, University of Tokyo for their guidance with regard to investigation of earthquake damage and planning of repair and strengthening work.

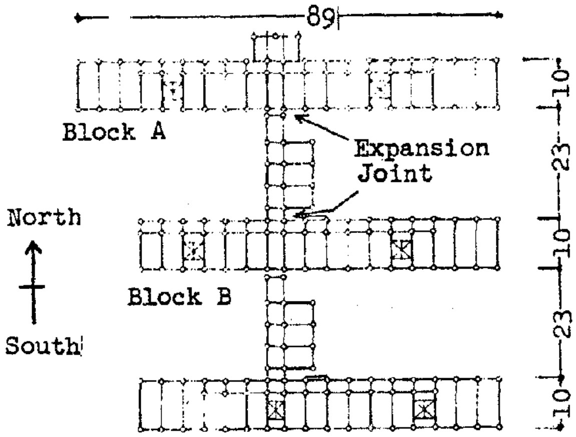
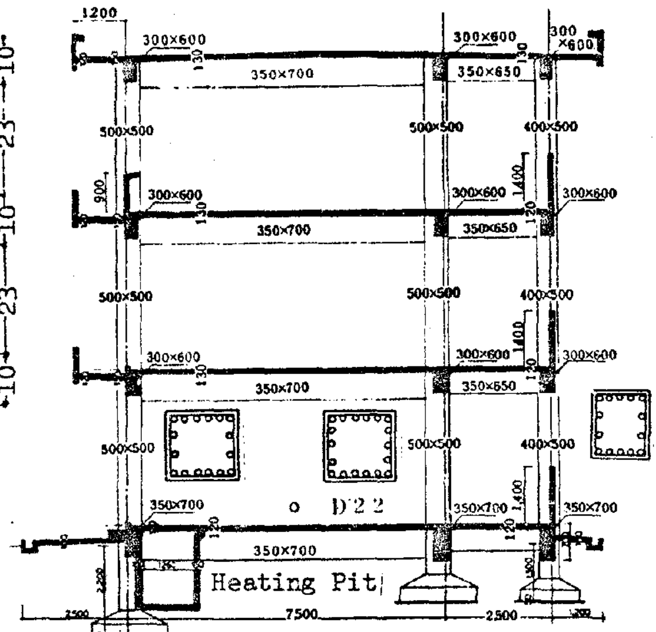


Fig. 1. Plan unit (meter)



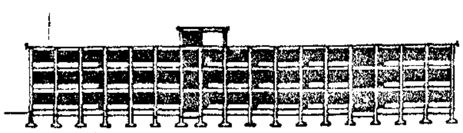


Fig. 2. South Elevation of Block C

unit (mili-meter)

Fig. 3. Section (Ordinary Classroom)

Table 1. Classification of Damage Levels of Columns

Damage Level	Description of Damage
0	No damage.
1	Slight bending cracks.
2	Large bending cracks or small shear cracks, crack widths not more than 3 mm.
3	Severe shear cracks, finish mortar completely spalled, crack widths more than 3 mm.
4	Severe shear failure, core concrete also failed, capability of carrying vertical load greatly reduced.

Photo. 1.
Damage Level 4
←

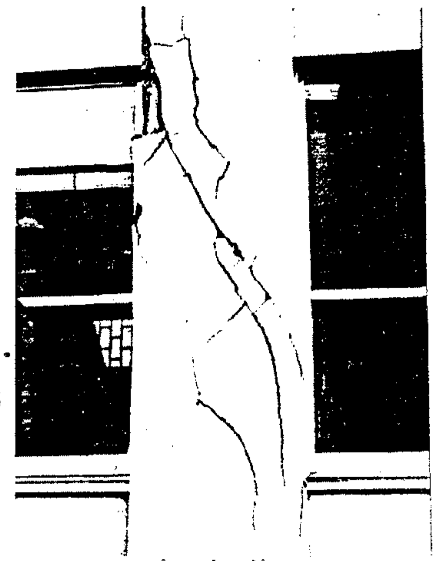


Photo. 2.
Damage Level 3
→

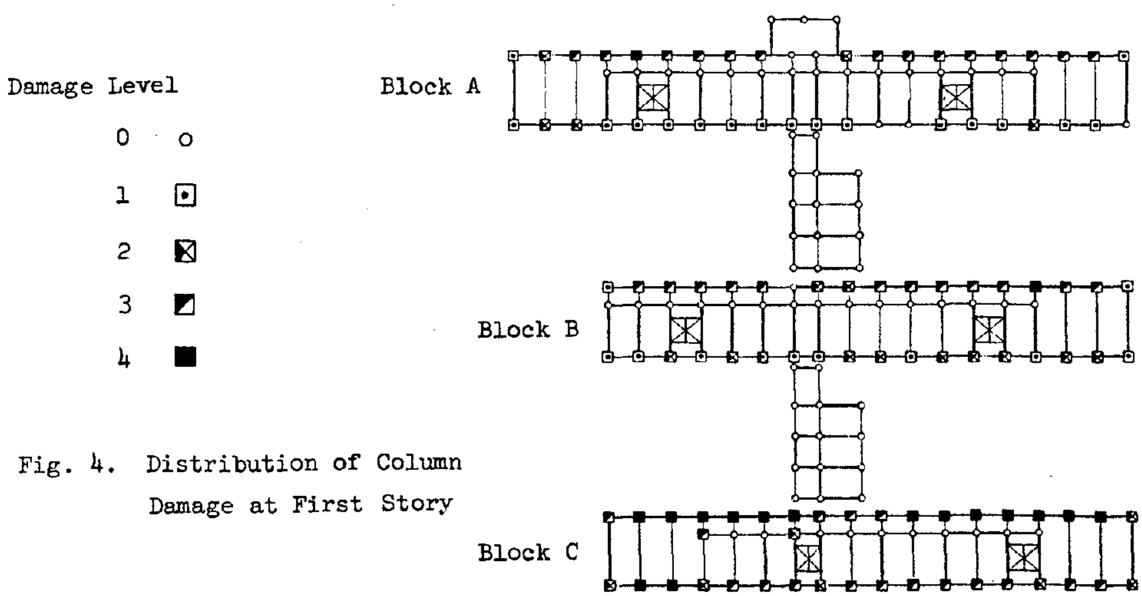


Fig. 4. Distribution of Column Damage at First Story

Table 2. Test Results of Steel

Block	Diam.	Yielding Stress (t/cm^2)	Tensile Strength (t/cm^2)
A	D22 Main Bar	3.70	5.63
B		4.08	6.27
C		3.64	5.70
A	9-mm Hoop	3.30	4.76
B		3.33	4.78
C		3.23	4.69

Each denotes the average of three pieces.

Table 3. Test Results of Concrete

Block	Floor or Story	Member	Compressive Strength (kg/cm^2)
A	1st	Column	239
	2nd	Beam	255-284
	3rd	Beam	236-256
	Roof	Beam	223-236
B	1st	Column	363
	2nd	Beam	230-239
	3rd	Beam	209-239
	Roof	Beam	334-352
C	1st	Column	218
	2nd	Beam	166-177
	3rd	Beam	195-204
	Roof	Beam	202-217

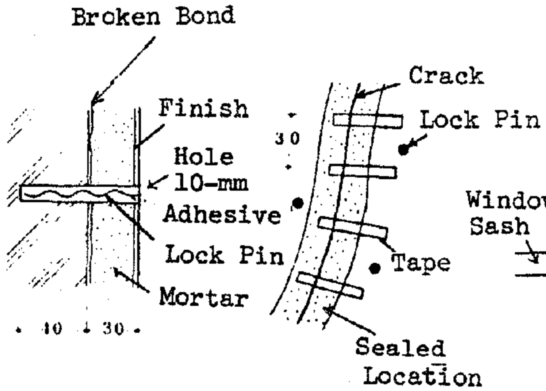


Fig. 5. Repair Method for Damage Level 1

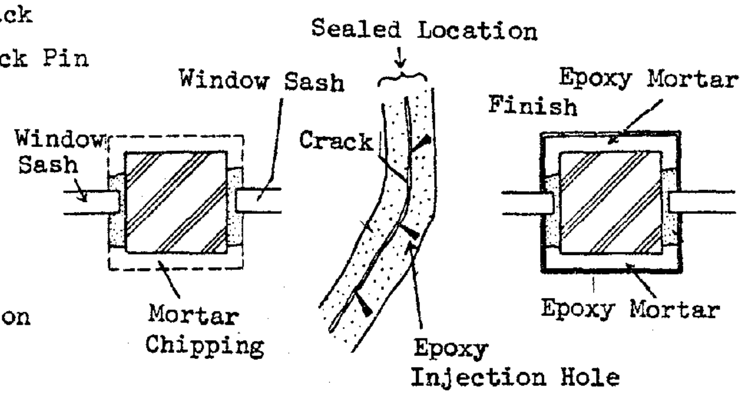


Fig. 6. Repair Method for Damage Level 2

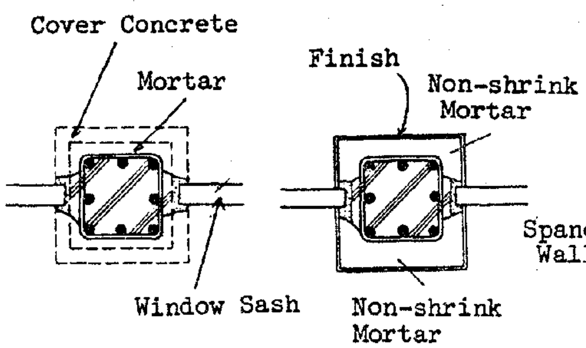


Fig. 7. Repair Method for Damage Level 3

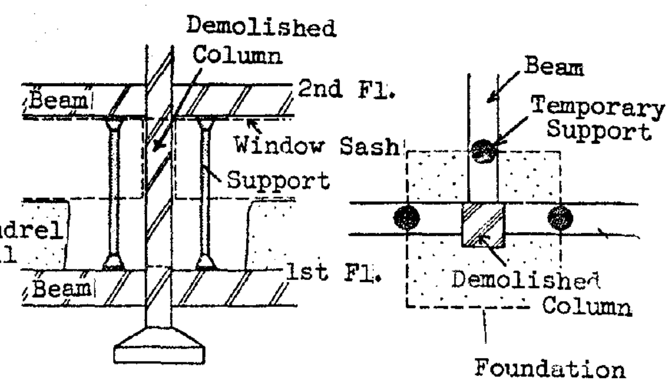


Fig. 8. Repair Method for Damage Level 4

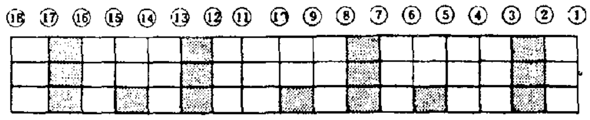


Fig. 9. Arrangement of Newly Provided Walls of Block C

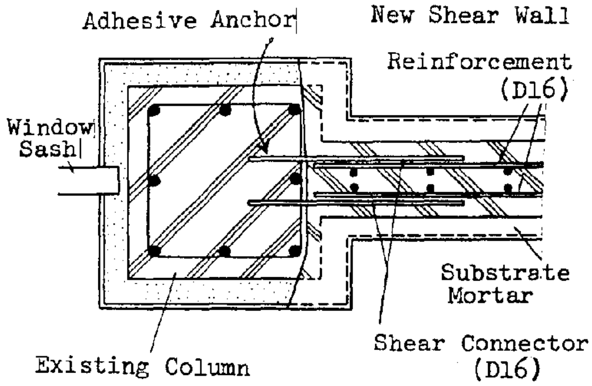


Fig. 10. Connection Between New Shear Wall and Column

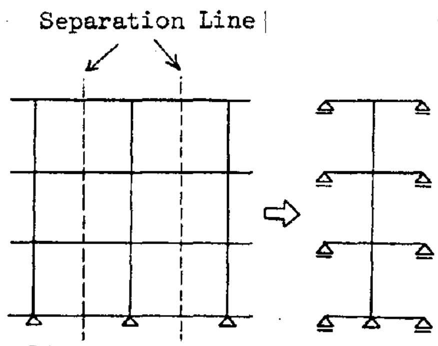


Fig. 11. Subassemblage for Analysis Model of Pure Rigid Frame

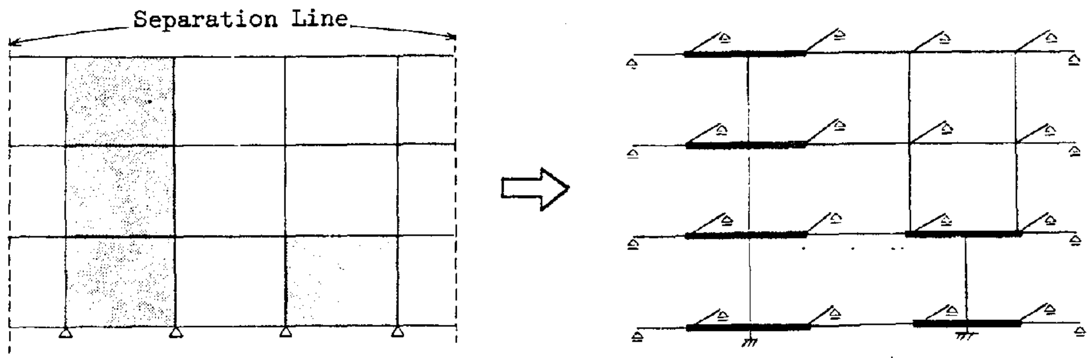


Fig. 12. Subassemblage for Analysis Model of New Shear Walls

Table 4 Horizontal Strength Possessed in Base Shear Coefficient at First Story in Ridge Direction

Block	A	B	C
Before Earthquake Damage	0.559	0.570	0.476
After Repair and Strengthening	0.745	0.760	0.785

SOIL MODIFICATION TO REDUCE THE POTENTIAL FOR LIQUEFACTION

by

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INTRODUCTION

A frequent contributor to damage or failure of structures during seismic events results from liquefaction of the soil materials underlying the foundation elements. To be subject to liquefaction, the soil must be submerged, and generally of a granular structure. Low density soils are the most sensitive to the phenomenon and the liquefaction potential generally decreases with an increase in density.

Because the potential for liquefaction is a function of the properties of the soil, reduction of such potential requires modification of the adverse soil properties. Because it is seldom possible to reduce the groundwater level, mitigating measures are generally limited to inhibiting the groundwater flow, increasing the soil density, or both. Recent experiences in the United States have utilized both Compaction Grouting and Vibroflotation to densify existing soils in place, and Chemical Grout Solidification to inhibit water flow through the soil.

Compaction Grouting

Compaction Grouting involves injection of stiff, mortar-like grout into previously drilled holes in the soil in a closely controlled manner. As the mass of grout increases, under pressure, the soil is densified through compaction. (figure 1) When the diameter of the grout column or mass is relatively small, the pressures are essentially radial and therefore horizontal. However, as the size of the mass increases, considerable up-lift force develops. It is surface movement caused by such upward force that generally control the quantity of grout placed at any given point. Masses of grout

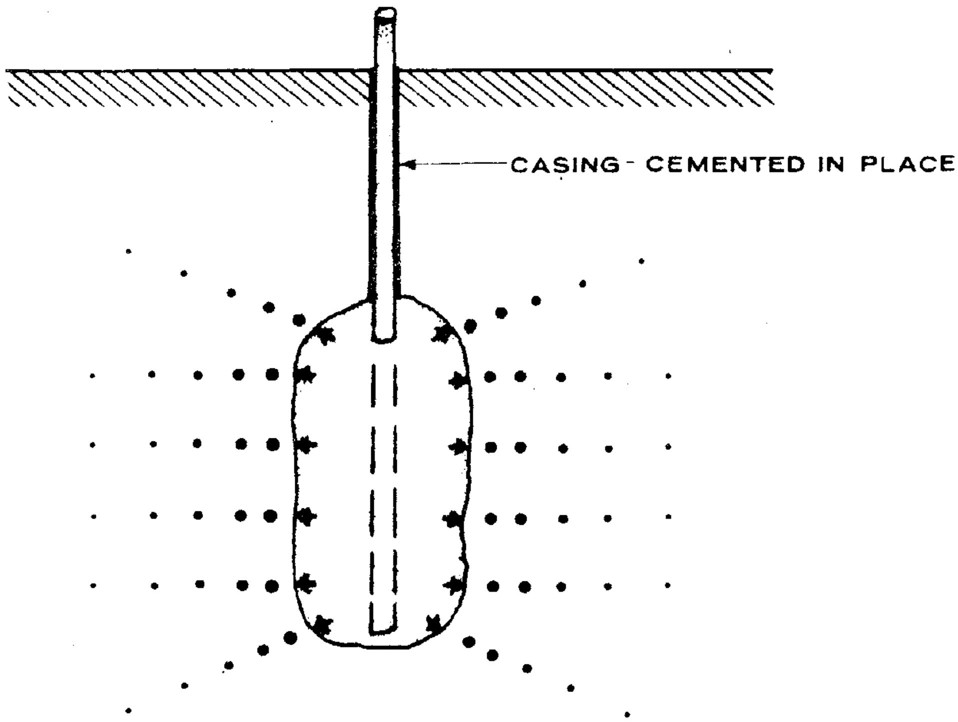

Figure 1. Typical "Growing" Grout Mass.

with cross sections of three feet or more are not uncommon.

The shape of the grout mass is usually spherical or cylindrical depending upon the amount of grout hole open during any given injection sequence. In uniform soils, the shape will be quite regular whereas extensive irregularities will prevail in non-uniform soils. The size of the resulting column will be affected by the existing soil density, moisture, and other properties, surface restraint conditions, and the injection pressure, rate, and other elements.

One of the principal advantages of compaction grouting is that its maximum effect is in the weakest soil zones. It is limited to soils and is most frequently used in materials finer than a medium sand. The process may be used in clays providing that adequate drainage is provided. Where drainage is limited to the extent that high pore pressures develop, much slower pumping rates are required and in extreme cases, the procedure is not applicable.

Compaction Grouting is particularily suitable to retrofit work in that it can be readily performed inside structures and other confined spaces, and its execution results in only minor interference in other operations as large equipment is not required in the immediate injection area. Although its most extensive use is in connection with settlement correction, it has been used specifically for densification of in situ soils to reduce the liquefaction potential. Amongst such projects have been the San Fernando Juvenile Hall repair following the San Fernando Earthquake of February 9, 1971.

Mechanics of Injection - The principal controlling factor in compaction grouting is the grout pressure behavior. It is usually monitored at both the grout pump and point of injection. Pressure behavior at the point of injection should be continuously monitored and recorded. This is unually done manually with a record entry being made at each significant change of pressure, however some work has been done in which continuous pressure recorders have been employed.

As aforementioned, injection is generally continued until a surface disturbance is noted. The pressure at which this will occur cannot be determined in advance and, in fact, it will vary widely between different holes of any given project. The key controlling criteria for the grout injection is to not permit too rapid a pressure build-up. The pressure level is eadily controlled by adjusting the grout injection rate. On most projects the initial injection rate will be on the order of 0.7 to 1.0 cubic feet per minute. If the pressure build-up is fairly slow the rate can be slowly increased. However, as the total volume of grout injected in the hole increases, the rate is generally lowered so as not to exceed the "optimal" pressure which has been determined for the specific job during the initial injection. Typical average injection rates will be on the order of 1.25 to 1.75 cubic feet per minute. Typical maximum pressure values will vary from less than 100 psi to 500 psi at the point of injection.

A great deal can be learned about the in situ conditions by the pressure behavior. Where the build-up is consistent, a relatively uniform soil is indicated. Where large fluctuations prevail, a very non-uniform condition is indicated. A sudden loss of pressure may be indicative of break-through into a void, such as might result from buried rocks or trash within a fill. A sudden loss may also be indicative of an impending surface disturbance, escape of the grout into an underground pipe or other substructure, or loss of lateral restraint such as might be provided by a retaining wall.

Grout Holes - Grout holes are usually about two inches in diameter. The spacing varies and must take into account structural restraint as well as soil conditions. Normally, holes are placed on about eight to twelve foot centers each way. As a rule, alternate "primary" holes are first grouted and their completion is followed by injection of the intermediate "secondary" holes. The holes are generally progressed in vertical stages of five to eight feet, working from the top down.

In practice, an oversized hole is drilled from the surface to the point at which stabilization is to begin, or a minimum of about four feet. A two- inch i.d. steel casing is then solidly cemented into the hole. The hole is then extended, working through the casing as required and the first stage grout injection made. At a later time, usually the next day, the hole will be extended through the same casing, for the next stage. This sequence is then repeated until the full depth has been reached.

As the compaction grouting process results in densification and therefore in considerable increase in weight of the treated soil, it is crucial to extend all holes to a very competent soil or rock zone. Attainment of such can be confirmed by extending an occasional hole an extra stage or two followed by injection. If indeed competent material has been reached, these extra stages should have very low grout takes and relatively high injection pressure.

The necessity for working from the top down has often been questioned, due to the greater expense involved by the increased drilling required. However, extensive experience and research has proven this to be a pertinent method. Because grouting of the upper stages increases their strength as well as density, their restraint capability is improved allowing greater pressure to be used in succeeding stages. There are occasions, however, in which it might be desirable to grout in a single stage from the bottom up. Such would be particularly appropriate when working at considerable depth in a fairly thin faulty layer.

Rotary drilling is the most common method utilizing either air or water to remove the cuttings. If the holes tend to close in, rotary mud can be used; however, it should be avoided if possible as any residual mud will act as an uncontrollable lubricant when the grout is injected. In some cases the holes are pre-treated immediately prior to grout injection. The most frequent such case would be injection of water where dry soils are involved. Such wetting ... will usually weaken the soil to be grouted facilitating its densification.

Grout Mixtures - Commonly used stiff, mortar-like grouts consist of fine "dirty" sand, portland cement, and water. The gradation of the sand materials is critical. If this componint is too coarse or the grains sharply angular, a harsh mixture will result. Under pressure, such a mixture would tend to have some of the water and cement pushed out and ahead of the remaining constituents resulting in high back-pressure and probable blockages. Too fine a sand material will result in an unstable grout. Optimum gradation for the sand material is indicated in figure 2. Sharp, angular sand should be avoided and well graded, rounded material favored. High clay contents, especially colloidal clays should be avoided as they will reduce the stability of the grout and may affect its durability when submerged. Admixtures though seldom used may be included where desired. The most common admixture is pozzolan which will reduce harshness if suitable sands are unavailable.

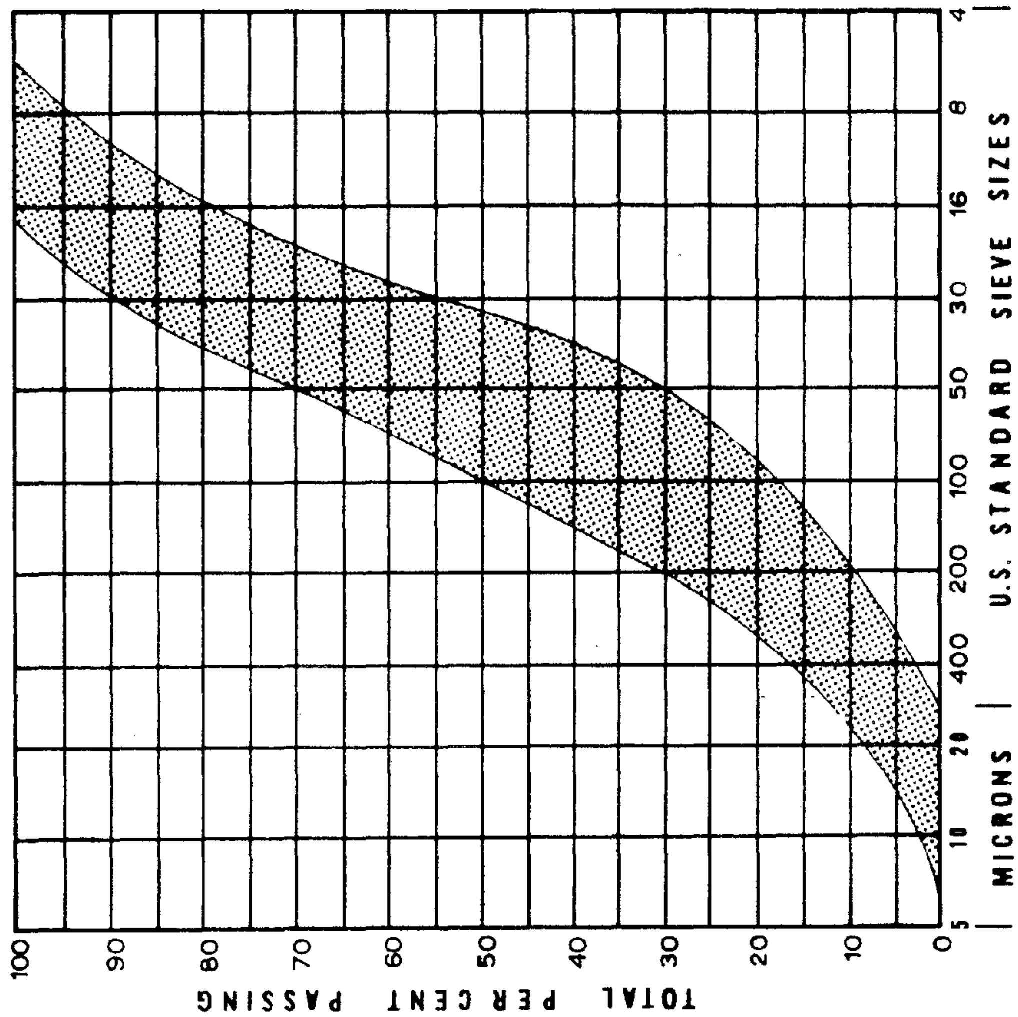

Figure 2 - Optimal Gradation for Sand Material

A very commonly used grout mixture contains about 12% cement and will provide an unconfined compressive strength on the order of four to six hundred psi which is more than adequate for most applications. Although such grouts have often been referred to as "no slump" grouts, it has become fairly acceptable to limit their slump to one inch using the standard concrete slump test ASTM C-143. In actuality, slumps of less than one and one half or two inches are probably acceptable. A good rule of thumb, however, is "the stiffer, the better."

Equipment - Because of the very stiff, relatively immobile characteristics of the grout material, special equipment is required for its handling. Conventional grout mixers and pumps used for "pourable" mixtures are not suitable.

Mixers generally are of the horizontal batch type with blades oriented to provide a chopping type action. Continuous mixers can be used but where employed must be provided with a metering supply system for the grout ingredients, in order to enable positive and uniform control of the grout consistency. Pump hoppers should be provided with an agitator which will force-feed the pump suction in order to prevent cavitation. The pump must be capable of handling the very low slump grout materials utilized. It should be capable of working at pressures in excess of 600 psi and preferably 1000 psi or more. Perhaps its most important requirement is the ability to operate at varying rates of displacement. Pumping rate must be controllable while continuously operating from virtually zero to about 2.0 cubic feet per minute. It also should be provided with some means of measuring the quantity of grout injected at any particular time.

Grout hoses are usually one and one half inch or two inches in diameter. Valves must provide a full flow opening. Any bends should be of a long sweep type in order to prevent major disruption of the grout flow. As with any type of grouting, suitable gage savers must be provided for the pressure gages which are usually located at both the pump and the point of injection. A two-way phone system or other communication medium should be provided.

Injection Control - As aforementioned, injection is usually continued until a surface disturbance is noted. Once surface movement has started, very little improvement will be made with continued grout injection and serious damage to the overlying, restraining soils can occur. Therefore, injection in any stage should cease immediately upon detection of minute surface movement. It is therefore imperative to carefully and continuously monitor the ground surface and any improvements thereon. When performing compaction grouting, personnel should give top priority to detection of surface movements and should employ any means or devices that will aid this detection.

A number of different devices are used for control. Perhaps the simplest and oftentimes most effective is a common string-line. Surveyors levels, laser instruments, multi-station manometers are other commonly used examples. The methods available to the engineer to monitor surface movements are virtually unlimited.

Records - As previously discussed, a great deal can be determined about the existing sub-surface conditions by careful evaluation of the grout behavior. Additionally, much information can be determined during the hole drilling. In order to be useful, however, such data must be systematically recorded. Specific important data which should be recorded includes the drilling method, length of casing, length of the grout stage, unusual drilling conditions such as encountering rock or organic materials.

During grout injection it is important to record the injection sequence of the hole stage, and sequential times, quantities, and pressures. Of particular importance is to note any drops in pressure during injection and, obviously, any surface disturbances should be noted in detail. The grouting records should be frequently evaluated in order to detect changes in the injection program which might be in order. Nearly all compaction grouting programs are designed as the work progresses and retrieved data evaluated; therefore, the importance of keeping good records cannot be overstressed.

Vibroflotation

Vibroflotation consists of achieving controlled densification of the soil through pre-planned sequenced penetration of the Vibroflote probe. The Vibroflote probe, (figure 3) is usually suspended from a large crane and is lowered so as to penetrate the soil while simultaneously vibrating and injecting water. A surface settlement crater results which is continuously filled with a granular soil material as the penetration continues. With proper performance, the settlement crater seldom becomes larger in plan than a few feet greater than the probe in diameter, and its prompt filling as created, prevents widespread areal settlement.

The primary limitations of the method for repair and retrofit work is the inability to use it inside most structures and the danger of damage to the structure due to the localized settlement that is inherent to the procedure. Where applicable however, it often offers cost advantages over grouting solutions, and is somewhat faster to perform. The procedure was successfully used immediately adjacent to an underground utility tunnel at Port Hueneme, California in 1981, wherein reduction of liquefaction potential of loose sands was required.

Chemical Soil Solidification

Chemical grout injection for the purpose of strengthening or impermeableizing soil has been practiced for over fifty years and in fact applications dating back into the 1800's have been reported in the literature. The procedure is applicable to the reduction of liquefaction potential and like compaction grouting has the advantage of being applicable inside structures or other confined locations. It was used in retrofit of Balboa High School in San Francisco, California in 1974. In this instance, the work was carried out from within a basement area with severely limited

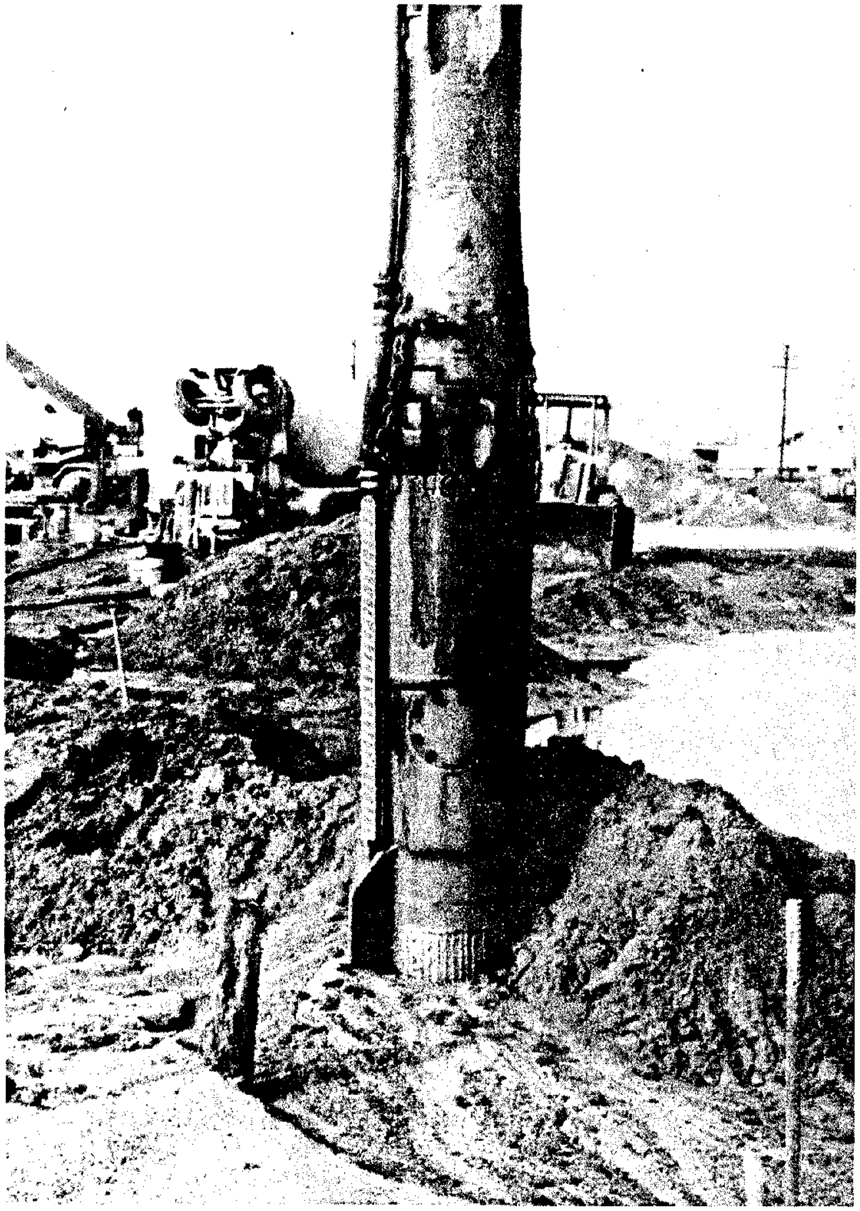

Figure 3 - Vibroflote Probe

headroom. The main limitation to use of the procedure are its applicability limitation to fairly clean sands only, and its very high cost.

The strength potential of a chemically grouted mass is dependent upon the type, and proportions of chemical grout used. Some chemical grouts when used in proper proportion will provide an essentially permanent mass, whereas others will begin to loose strength some time after injection or exposure to the elements. Also it is important to note that many factors effect the strength of the grouted mass and in most cases, the actual usable strength value is considerable less than indicated by standard test methods such as the unconfined compression test. Because of the above, conservative design and the use of high factors of safety are always appropriate.

In addition to the grout materials chemistry, the strength of a grouted mass is influenced by a number of different, although often controllable factors. This subject was extensively discussed by Warner (4) wherein the results of an extensive laboratory and field research program were reported. Some of the more important factors therefrom are summarized as follows:

Curing Enviroment - Most chemically grouted masses attain appreciably higher strengths when the grouted specimens are allowed to dry out. It is therefore important to perform any strength evaluation tests at the natural moisture content of the mass.

Incremental Loading - The rate of loading has an effect upon the strength that a grouted mass will obtain. In general, slow incremental loading will result in appreciably higher obtained strength.

Effect of Continuous Load - Most chemical grouts are subject to creep and therefore exhibit far lower strengths under continuous loading. The load at which a specimen can withstand without further strain is usually referred to as the "fundamental" strength. It is usually less than 50% of the ultimate strength determined by the relatively rapid loading of the standard unconfined compression test. Figure 4, indicates the ultimate and fundamental

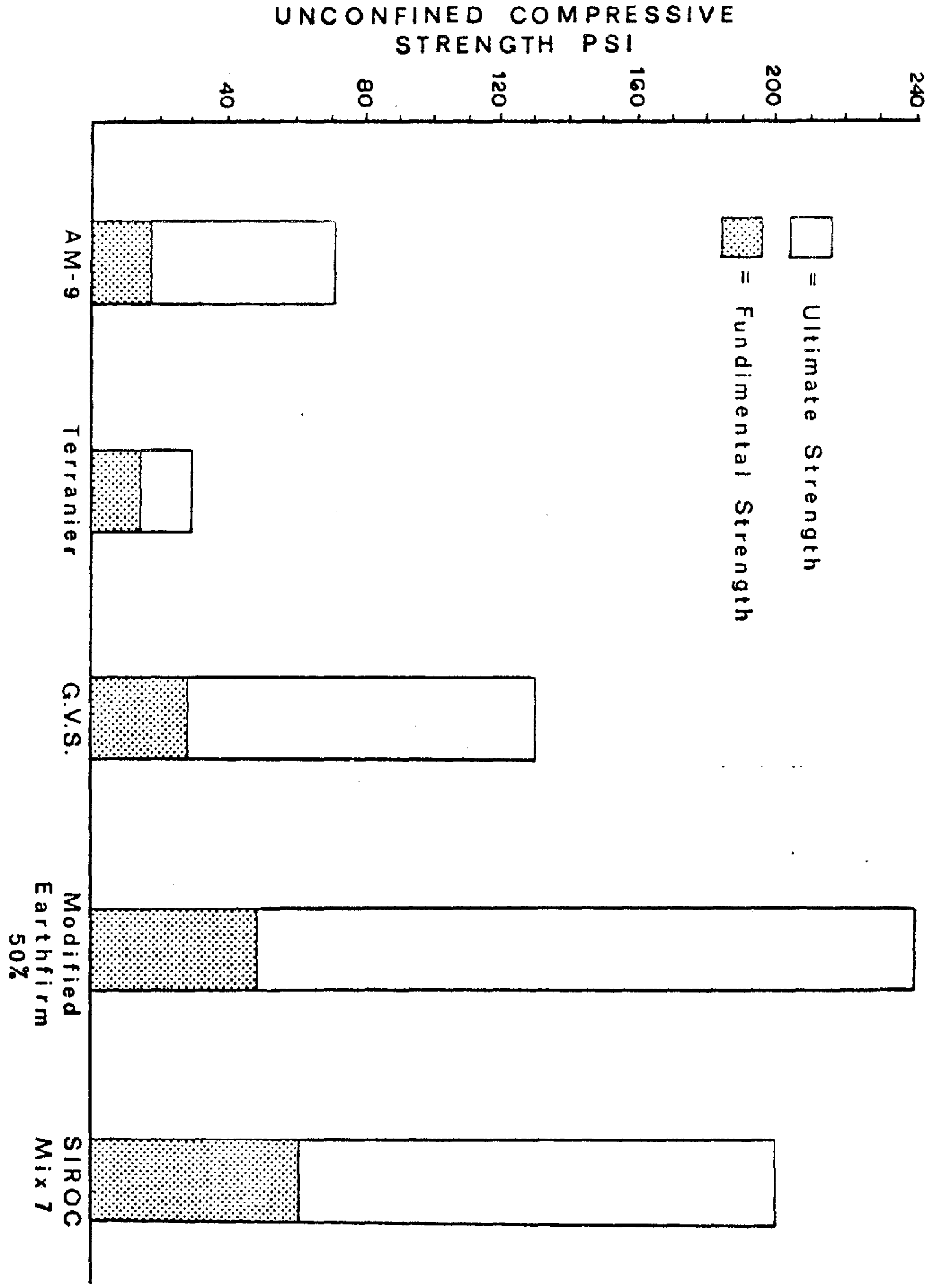

Figure 4. Ultimate and Fundamental Strength of Chemically Solidified Soil.

strengths of solidified specimens utilizing five different chemical grouts. Therein, the G.V.S., Siroc Mix 7, and Modified Earthfirm all utilized sodium silicate as the base material and each contained 50% by volume thereof.

Wet and Dry Cycles - Only limited data is available upon the effect of wet and dry cycles on chemically solidified masses. However, based upon available data, most chemically grouted masses loose strength under such enviromental conditions.

Stress-Strain Relationship - All chemically grouted masses are subject to strain upon loading. The rate and manner of loading (uniform or incremental) both influence the total amount of strain. In data presented in (4), creep strain with a magnitude of 4% to 9% were experienced with silicate based grouts. This compares to nearly 20% for Acrylimide grout.

CONCLUSIONS

Repair or retrofit efforts in order to prove effective in future seismic events must provide for stable foundations. When the particular structure is located on a soil material that is subject to liquefaction, modification of that soil must be made in order to reduce the potential to an acceptable level.

Recent experience in the United States has involved use of compaction grouting, vibroflotation, and chemical soil solidification in this regard. Compaction grouting has the advantage of being used inside structures and other confined locations. It also can be used where occupancy of a structure must continue as it involves minimum disruption during progress. Its main limitation is lack of applicability in clean medium or coarse sands. Vibroflotation is faster and less costly than grouting, however it cannot be performed from inside or under most structures, and damage can result from the probe settlement craters, if the structure lacks sufficient rigidity. Chemical soil solidification is the most costly method, however it is effective in clean sands and like compaction grouting can be performed in congested areas and with a minimum of disturbance to other activities.

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VETERANS ADMINISTRATION SEISMIC CORRECTION PROGRAM

Richard D. McConnell*

Summary

Following the San Fernando Earthquake of February 9, 1971, when two patient occupied buildings of the Veterans Hospital at that site collapsed killing 46 persons, the VA was required to undertake a full seismic program. This program included the appointing of an Advisory Committee, preparation of a seismic code and a program to update all VA Facilities to prevent such a recurrence. To implement these directives, a program of site evaluations to determine the predictable seismic levels was undertaken at 68 hospital sites. Following the site evaluations, most of those sites were then investigated as to the adequacy of the structures at those locations to withstand the predicted level of possible seismic events. As a follow-up phase to the first building studies, a subset of the structures investigated, which had been found to be deficient, were studied as to alternate means for adequate hardening to bring them to the required seismic strength. In the years following that San Fernando Earthquake, VA has undertaken alterations to harden deficient structures at many of these sites. In addition, buildings were demolished or occupancy changed where deemed necessary. The VA Seismic Code Standard H-08-8 has been used for the past seven years for the design of all new hospital structures and is the criteria for all retrofit. An instrumentation program was introduced and studies were undertaken and requirements issued concerning nonstructural details; post-operational utility provisions; control of Furniture, Equipment and Supplies; and related studies such as masonry testing, soil stability, etc.

Introduction

The San Fernando Earthquake of February 9, 1971, destroyed or severely damaged four major hospitals, including two patient-occupied buildings of VAH, San Fernando, that collapsed, killing 46 persons. The VA buildings were designed and constructed prior to the development of seismic design codes. The other non-VA hospitals were of relatively recent construction and were designed to resist earthquake forces. None of those hospitals could be salvaged.

A House of Representatives Subcommittee on VA Hospital Disaster held formal hearings on February 22, 1971 and received testimony from the Deputy Administrator, other VA officials familiar with the disaster, and several technical experts. In an Interim Report dated March 18, 1971, the Subcommittee made several recommendations. One key recommendation was the following:

> "The Veterans Administration should immediately identify all structures in the VA hospital system located in hazardous zones to determine what must be done to make them safe. If modernization of these facilities to meet latest seismic and other safety standards is not considered feasible, either replacement structures should be built or patients relocated in safe structures at other hospitals."

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The Veterans Administration appointed a Committee of consultants who developed new requirements for earthquake resistant design of VA hospital facilities. In December 1973, an Advisory Committee on Structural Safety of VA Facilities was appointed, as required by Public Law 93-82, to recommend standards for fire, earthquake and other natural disaster-resistant construction. The Advisory Committee formally recommended the earthquake-resistant design requirements developed by the earlier Committee and they have been adopted.

VA Seismic Code

The Committee decided that the VA design code should be based on the Earthquake Regulations of the Uniform Building Code, but with some major modifications. Those UBC Earthquake Regulations are the most widely accepted such code in the country and are revised on a regular basis by the Seismology Committee of the Structural Engineers Association of California. The principal modifications we made were:

1) We are not using code level forces. We evaluate our sites for geologic and seismic hazards, and design new facilities to resist the strongest ground shaking that can be expected at the site. Existing structures were evaluated in terms of the maximum ground shaking that may reasonably be expected during the planned life of the facility.

2) The Uniform Building Code did not include at that time the effects of soils on earthquake motions, although, these effects may be important. We retained Professor Matthiesen, University of California at Los Angeles, (now with USGS), and Professor Scott, California Institute of Technology, to prepare short summaries of present knowledge in (a) amplification of strong ground motion due to soil layering, (b) soil structure interaction, (c) soil liquefaction, and (d) slope stability and the design of retaining walls.

3) Considered ductility/damping factors for different types of structures.

4) Modal method dynamic analysis for certain classes of structures.

5) The Uniform Building Code required reinforcing of all masonry that resists seismic forces. This is quite reasonable for the West Coast areas where both the level of ground shaking and the frequency of earthquakes are high. However, masonry in the great majority of existing VA structures in other areas of the country was not reinforced. We asked the Bureau of Standards to: (a) investigate tests of failure of masonry induced by earthquake loading, and (b) propose test methods for determining the strength and stiffness properties of existing masonry. Out of this, we hoped to develop safe lower bounds of masonry strength that could be used to evaluate our existing structures realistically.

Masonry Testing

In the beginning of the masonry testing program, sampling and testing of the wall specimens were conducted generally in accordance with the National Bureau of Standards recommendations. This entailed cutting a wallette approximately 1-1/2 foot square out of the existing building walls and testing pplying a compressive load across the diagonal. The cost per sample was approximately $2,000 including the sample cutting, testing and patching of the hole. It should also be mentioned that such an operation almost necessarily causes some disruption in hospital operations.

It was decided that something less expensive and less disruptive was necessary. For this we turned to cylinder cores taken from the walls. A problem is encountered here because the results of cylinder tests do not quite correlate with the resultls of wallette tests.

Testing Engineers, Inc. undertook to examine this problem. In the test program which was run by Mr. F. R. Preece, shear strength values were obtained from wallette samples and from cylindrical core samples. Mr. Preece found that close agreement between the two methods was reached by including the influence of internal friction.

The cylinder core sample has two big advantages over the wallette type sample. First, the cylinder core costs only approximately $200 per sample versus the approximately $2,000 cost per wallete sample. Second, a cylinder core can be taken and the hole repaired relatively quickly, with a minimum of disruption of hospital functions, whereas cutting and replacing a wallette sample is disruptive.

The values of ultimate tensile and shear stresses are translated into allowable stresses (via the NBS approach, which has been incorporated as part of the VA's Standard H-08-8). These allowable stresses then determine whether an existing wall must be reinforced or whether it is a viable structural element (shear wall) which is capable of resisting shear forces.

The results of the Veterans Administration program of testing wall cores, to no one's surprise, can be characterized as extremely varied. Shear ultimates ran from almost zero to well over 300 psi. These included sample results from brick, concrete block and clay tile. The samples were from all over the country and represent almost the full range of age and workmanship that can be found in unreinforced masonry.

During the progress of the seismic program, the VA has found that walls that might look at first to have no strength and thus no influence on building reaction to earthquake forces are, in fact, quite strong and therefore constitute major load paths.

The VA also found that even when the walls are understrength, a strengthening procedure that has come to be known as the Boise method (building another wall on the outside of the old one) can economically ($15 to $30/ft.2) strenthen old buildings.

Program Description

Sixty-eight of the existing VA Hospitals are in geographical areas where moderate or major earthquakes have occurred. Most of the 68 hospitals have some unreinforced masonry. The buildings in these hospital centers range in age from 6 or 8 years old to well over 70 years. The total money investment is in the billions of dollars and, of course, the replacement cost would be many times the original cost. Consultants were retained to study the seismic and geologic hazards at each site. These reports were reviewed by the U. S. Geological Survey and NOAA to confirm the evaluation of the seismic risk.

It was decided that the deficient VA hospitals would be strengthened to withstand any earthquake that might be expected in their areas in the next 100 years. This involved a structural evaluation of many of the 68 hospitals, and each hospital usually has many buildings. Other consultants evaluated the ability of the buildings to withstand earthquake forces and provided alternate procedures with corresponding cost estimates for those buildings found to be in need of strengthening.

All substandard buildings essential to the operation of medical centers in areas of high seismicity have been or will be corrected as necessary. Some have been demolished. It was decided that, in areas of moderate seismicity, only substandard bed patient buildings would be corrected. Also, correction of nonstructural deficiencies that present potential life hazards or threats to facility operations would be included in these projects.

Seismic construction projects have been programmed along with other necessary corrections to maintain a balanced annual construction program within available resources and consistent with other system-wide priorities.

In addition to basic design standards, the VA has developed seismic requirements for utility services, architectural components, and equipment and furniture.

The Veterans Administration has had a Strong Motion Instrument Program for ten years. This program involved the purchase of 66 instruments which were placed at 56 stations around the entire country. These instruments were installed by NOAA and USGS and maintenance is funded by the VA and performed by USGS. It is interesting to note in this regard that the Jan. 24, 1980 Livermore earthquake which sustained significant damage in facilities other than Veterans Administration had only one set of instrument records which could be used for post-analysis. These records were obtained at the Veterans Administration Hospital at Livermore. Those records, incidentally, showed peak values of .17g at the basement level and .64 at the roof level.

Accomplishments to Date

The most serious problems encountered by the VA were in California. It was evident after receiving reports on the condition of the buildings and the cost of required reinforcement that the agency had no choice but to evacuate potentially hazardous facilities at Wadsworth, Menlo Park, and Livermore. A large percentage of buildings at these West Coast Hospitals were constructed before 1935, when no seismic design requirements were in general use, and were found to be unable to withstand earthquakes if high intensity. The inspection of all 22 buildings at the San Francisco hospital indicated no need for relocation of patients, although minor alterations would be made in some of the buildings, and the boiler house was relocated.

Of the 82 buildings at Menlo Park Division of the Palo Alto Hospital, 18 were deemed structurally unsatisfactory in event of a major earthquake in the area. But of those 18, only one was actually occupied by patients.

At Livermore, 13 of the 48 buildings were pinpointed as potentially hazardous, including the largest patient-occupied building on the station.

Of the 236 buildings at the West Los Angeles complex, 30 were graded potentially hazardous, including eight patient-occupied structures.

The remaining four VA hospitals in California -- as well as the Palo Alto Division of the two-hospital operation at Palo Alto -- are of more recent construction, and were judged to be of satisfactory seismic construction. However, a new review of some buildings at these sites is now in progress.

In the face of such circumstances, the VA acted immediately to vacate the weaker buildings and to start the improvement of those which had to be continued in service. An extensive program was necessary to relocate patients and staff from the weaker existing buildings to new and converted facilities at a number of other hospitals. Replacement hospitals at Los Angeles and Loma Linda provide modern facilities for those destroyed by the earthquake or evacuated because of structural weaknesses. Another major building to replace seismically deficient units is programmed for Palo Alto. Evaluation and strengthening of other facilities is continuing where necessary.

To this date, the Veterans Administration has spent more than 250 million dollars in the total seismic correction program including site studies (site prediction and review of buildings), demolition, structural and non-structural hardening, temporary facilities and procedures for the post-earthquake California relocation plan, and three replacement hospitals.

The Veterans Administration Program for seismic deficient facilities is confronted with numerous concerns and problems which may not be evident to all at first. One of these is the requirements by Historical Preservation Groups at various government levels. This has been most significant in correction programs which have involved modifications to exterior walls at a number of our sites.

Another major concern is disruption. This becomes a severe problem for VA hospitals where occupancy and functions must be sustained.

More recently, and in direct response to geologic evidence that a major earthquake has a better than 50% probability of occurring within the next decade, the VA has begun programs to: (1) anchor major mechanical and electrical equipment so it will not be dislodged and disrupted during an earthquake; (2) install an emergency radio network to facilitate direct communication between any VA facility in California and any other in the United States; (3) provide for emergency utilities, especially water and electrical power, in all VA medical centers; (4) provide heliports at all existing medical centers for emergency use; (5) install special earthquake provisions for equipment, furniture, and supplies to assure that VA Medical Centers can function as a community resource in the post-earthquake period; and (6) conduct earthquake drills.

Examples

The following two page Table presents a recap of 13 stations which are recent projects for seismic hardening under design or construction. This Table indicates the munber of buildings, cost, the current status and a brief statement of the method of hardening projects. Selected details for 6 of these sites follows. The purpose of these details is not primarily to show specifics such as dimensions in all cases, but to present the variations of types of hardening used and also provide a qualitative description, by way of the details, to indicate some of the problems and how they are resolved.

In general, two of the stations, Boise and Salt Lake City principally used what we now classify as the "Boise Method" in that the exterior shear walls are new concrete placed on the outside of the existing exterior wall and new face brick placed on that concrete.

In contrast the shear walls on Charleston on the exterior are to have shotcrete and shaping such that the concrete will be exposed.

On American Lake and Walla Walla, most of the shear walls were exterior shotcrete applied and essentially left without face brick.

Prescott was distinct in that it used 4" shotcrete on the inside of the exterior masonry walls due to Historic Preservation objections to any exterior alterations.

Fort Harrison used interior reinforced concrete shear walls.

Fresno will use a combination of interior steel K bracing, concrete and steel shear walls as well as some exterior shear walls. In addition to these generalizations, there were some variations of interior/exterior types in various projects.

It should be pointed out that in all cases every effort was made to minimize disruption by accounting for the lateral strength required by incorporating as few interior shear walls as possible in the plan. Most of these western stations had difficulties meeting historical preservation requirements in regard to finish.

It can be noted on the Salt Lake City and Boise details that the so called "Boise Method" incorporated the use of shear keys imbedded into the existing masonry in conjunction with reinforcing ties to both the existing masonry and the new outside wythe.

Originally at Salt Lake City, the intention was to peel away the outer face brick and then proceed with the concrete shear portion and a new outer wythe. It was found however, that there were two problems. One was the substantial bond in the masonry and the second was the factor of height of some of the buildings.

The Ft. Harrison details show a means of adding stiffness to the columns and also indicate the steel bracing.

The detail for the exterior walls on American Lake shows the type of "Key Pocket" as it is called and also shows the outside stuco plaster finish on top of the concrete shear portion.

The details of Prescott are uniquely interesting because that was timber existing construction.

Although there are no details for it shown in this section, the hardening at West Roxbury Center is distinct in contrast to the examples shown here in that it provides new wings which will be stitched to the existing buildings thereby buttressing the entire combined unit for lateral forces.

For the Charleston project, steel cross bracing and the use of end towers were dismissed for various reasons and, ultimately, four types of shear walls were developed: one story reinforced concrete shear walls in the pipe basement; exterior reinforced concrete shear walls; interior reinforced concrete shear walls; and interior steel shear panels.

As is noted in Figures 7 and 10, walls above the third floor in the longitudinal direction are exterior reinforced concrete while those in the transverse direction are interior steel shear panels. Below the third floor, the exterior walls become interior reinforced shear walls on the South side.

Figure 8 shows a transverse section through the building after new shear walls will be added. In contrast to the "Boise Method", on this project, the outside brick and insulation are to be removed before installation of the concrete shear wall.

Figure 9 shows a typical cross section of the existing and modified exterior wall. The use of the steel shear panels was to minimize construction disturbances, they either to be built in place or prefabricated as units and connected in place. Since they are to be located against existing walls, these new steel shear panels were designed so that all welded connections are to be made on one side only. Figures 11 and 12 show the steel walls.

The following hardening costs are on a square foot of gross area basis and are indexed to April 1981 dollars:

Boise	$22 psf
Salt Lake City - Bldgs 1 thru 5	$15.30
Salt Lake City - Bldgs 6 thru 9,27 and 28	$19.60
Charleston	$22.23

The cost per square foot of wall area on Salt Lake City, Buildings 1 through 5, (indexed to April 1981) was $17.70.

Acknowledgements

The author would like to thank Mr. Mohammad Ayub, Structural Engineer with the Veterans Administration for his assistance in the preparation of this paper. Also, he would like to thank Mr. Joseph Baldelli of the firm of John A. Blume and Associates for his willingness to permit the use of some of his sketches of the Charleston project to be used in this paper.

ACTIVE SEISMIC PROJECTS UNDER DESIGN OR CONSTRUCTION

May 6, 1981

STATION	g	PROJECT#	BLDG.NOS	COST Estimate	COST Bid	PROJ.STATUS	METHOD OF HARDENING
AMERICAN LAKE, WA	0.2g	505-042	4		926,600	Construction Complete November '77	Use of 4"-5" shotcrete on outside of masonry wall
AMERICAN LAKE, WA	0.2	505-063	5	$1,400,000		Prelim. dwg to start soon	
AMERICAN LAKE, WA	0.2	505-053	7		$500,000 (Approx.)	Construction in progress	Use of 4"shotcrete on outside of exterior masonry walls
BOISE, ID	0.15	513-032	13,67,27, 77,34,2, 28,29,8,79		$2,391,373	Construction Complete December '76	Exterior conc.shear walls on perimete of bldg.plus new fascia brick
CHARLESTON, SC	0.25	534-016	1		$8,584,000	Construction to begin soon	Exterior conc. shear walls on perimeter of bldg plus some interior conc. & steel shear walls
FT. HARRISON, MT	0.30	436-038	16,43,47, 141,142, 150		$1,249,100	Construction Complete April '80 -(Station Level)	Interior conc. or reinforced masonry shear walls
FRESNO, CA	0.23	570-016	Main Hosp	$4,100,000		Working Dwg in Progress	Interior steel K bracing & some exterior shear wall at selected bays
LOS ANGELES, CA	0.25	691-093	113	$ 800,000		Station Level Job (Under Construction)	Exterior gunite shear walls at perimeter of bldg.

NOTE: "Construction Complete" indicates substantial completion in accordance with base bid.

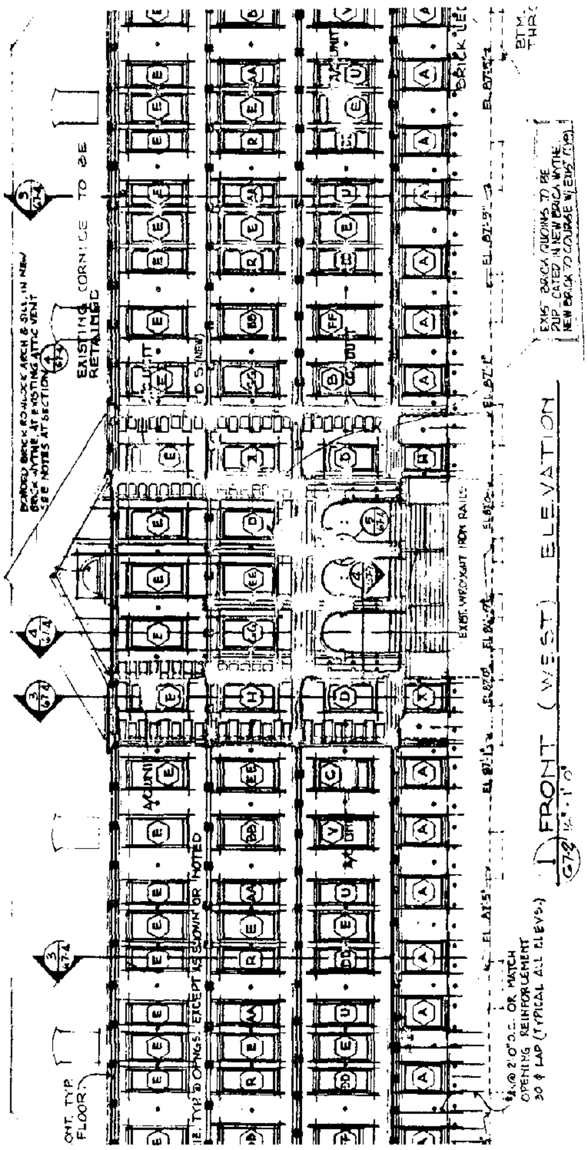
SEISMIC PROJECTS TABLE - PART A

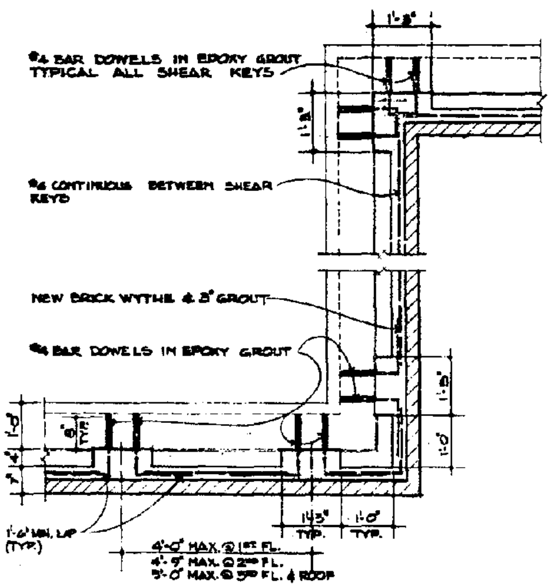
ACTIVE SEISMIC PROJECTS UNDER DESIGN OR CONSTRUCTION

May 6, 1981

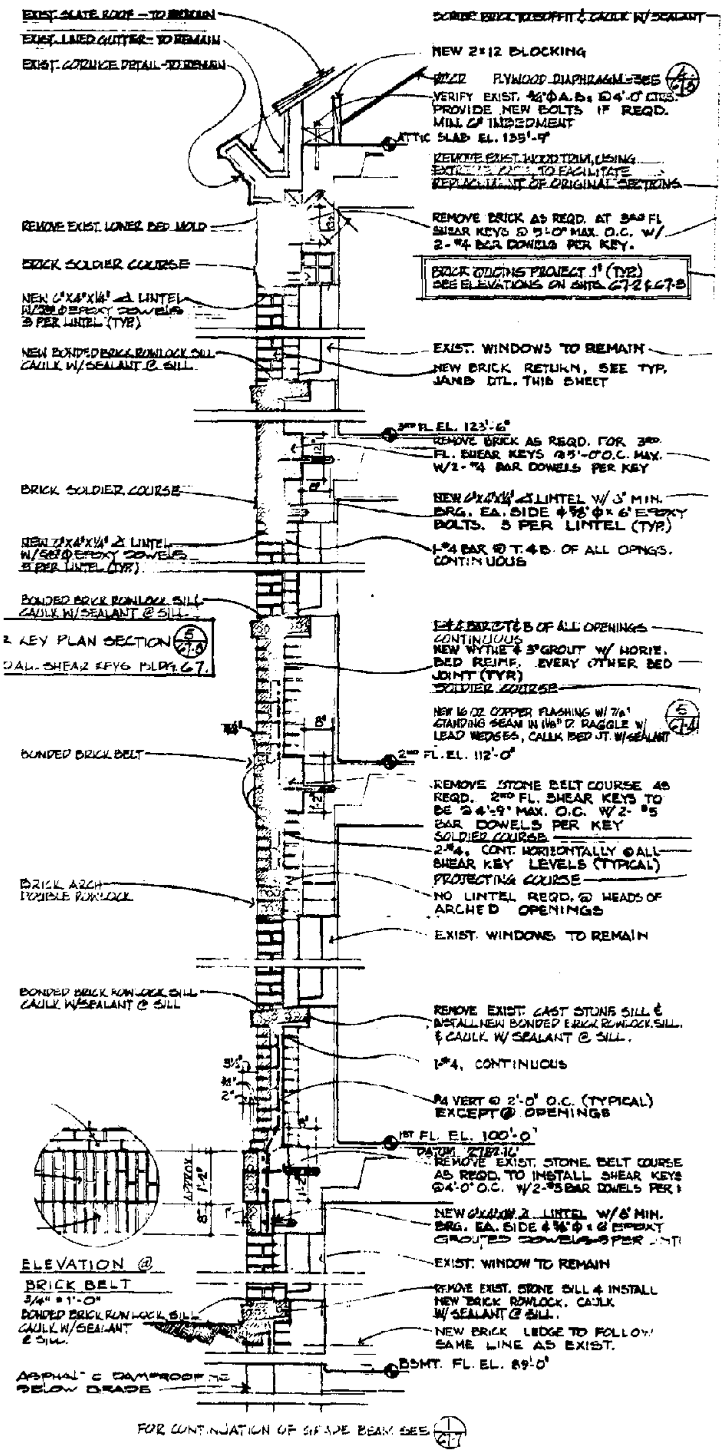
STATION	g	PROJECT#	BLDG.NOS.	COST Estimate	COST Bid	PROJ.STATUS	METHOD OF HARDENING
OKLAHOMA CITY, OK	0.10	635-018	1a,1b,1c, 1d	$4,000,000 (Approx)		Working Dwg Complete Out for Bid	Exterior conc.shear wall at selected bays only plus new fascia brick
POPLAR BLUFF, MO	0.25	647-018	1,5,6,8	$10,550,000		A/E to start WD soon (Prelims Complete)	Likely method will be exterior concrete shear walls at perimeter plus new brick
PRESCOTT, AZ	0.15	649-048	15,17,19, 20,42		$ 600,000	95% Construction Complete (Station Level)	Use of 4" shotcrete on inside of exterior masonry walls.Some interior shear walls too
SALT LAKE CITY, UT	0.30	660-019	1,2,3,4, 5		$6,413,650	Construction Complete October'79	Exterior conc.shear walls on perimeter of bldg plus new fascia brick
SALT LAKE CITY, UT	0.30	660-022	6,7,8,9, 10,13,27,28		$2,320,650	Construction Complete April '79	Exterior conc.shear walls on perimeter of bldg plus new fascia brick
SAN FRANCISCO, CA	0.30	662-037	2,4,200	$1,350,000 (Approx)		A/E to start WD soon (Prelims Completed)	Likely method will be interior shear walls plus strengthening of diaphragm & spandrel beams
WALLA WALLA, WA	0.15	687-046	66, 86		$ 933,000	Constr. in progress	Exterior conc.shear walls(gunite) at a few selected bay & interior conc. shear walls
WEST ROXBURY, MA	0.10	690-024	1	$13,600,000 (Includes other works)		Working Dwgs. Complete	Addition of 4 conc. shear towers & some exterior shear walls. Stitched existing expansion joints.

SEISMIC PROJECTS TABLE - PART B





5 TYPICAL PLAN SECTION
67-8 3" = 1'-0"

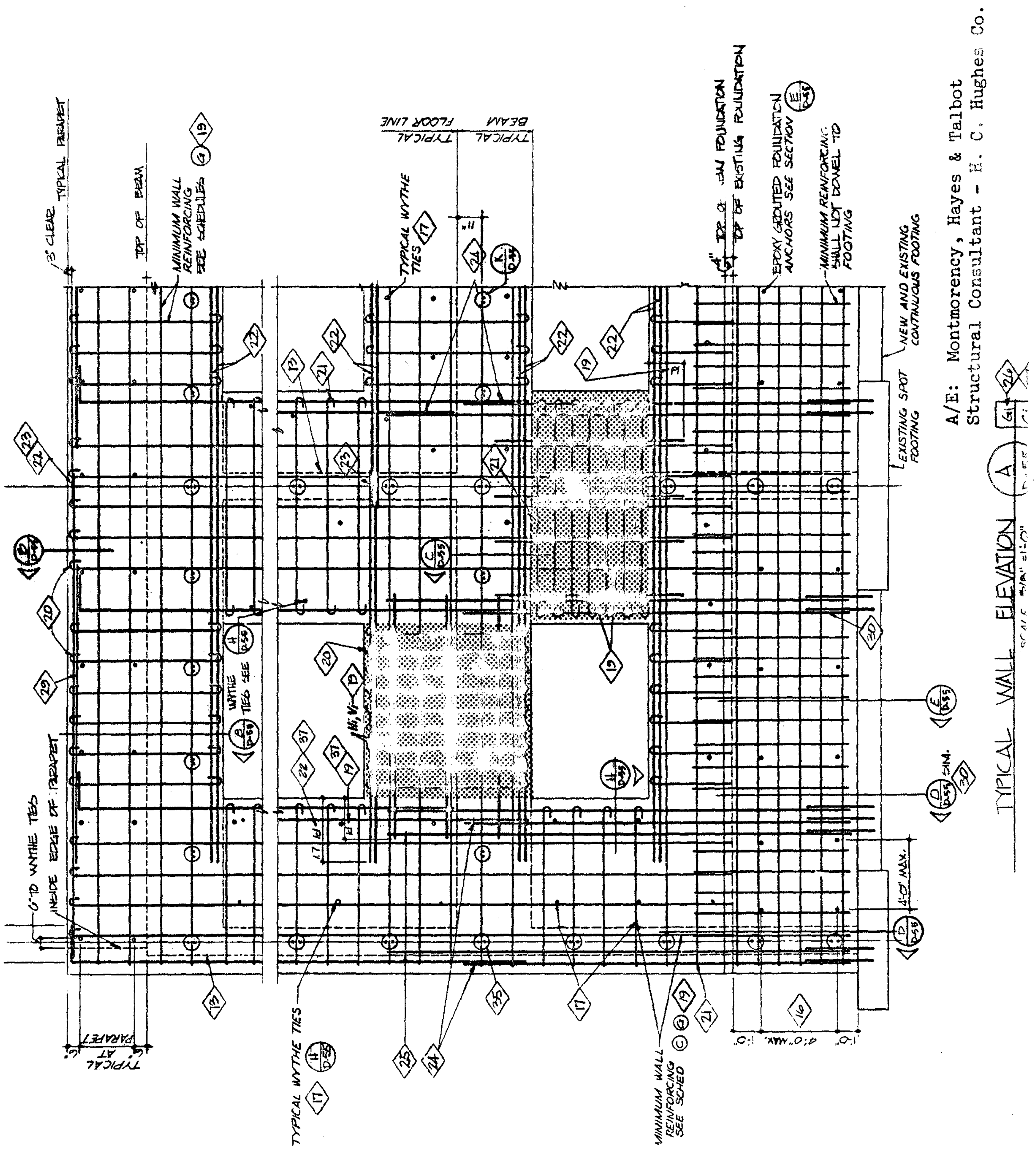


3 WALL SECTION
67-4 3/4" = 1'-0"

BOISE, ID

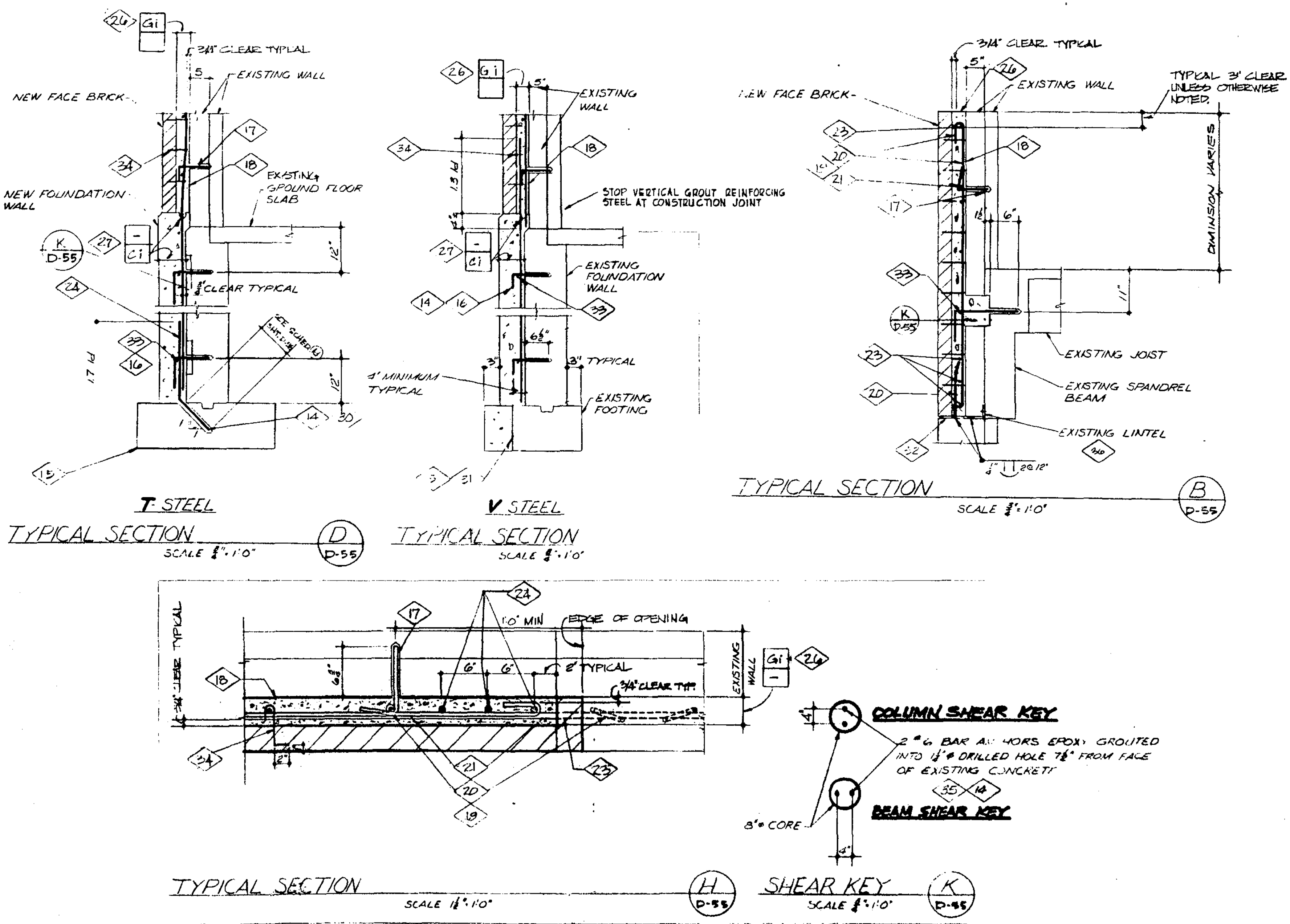
Figure 1

Walter Kidde Constructors, Inc.,
Englewood, Colorado



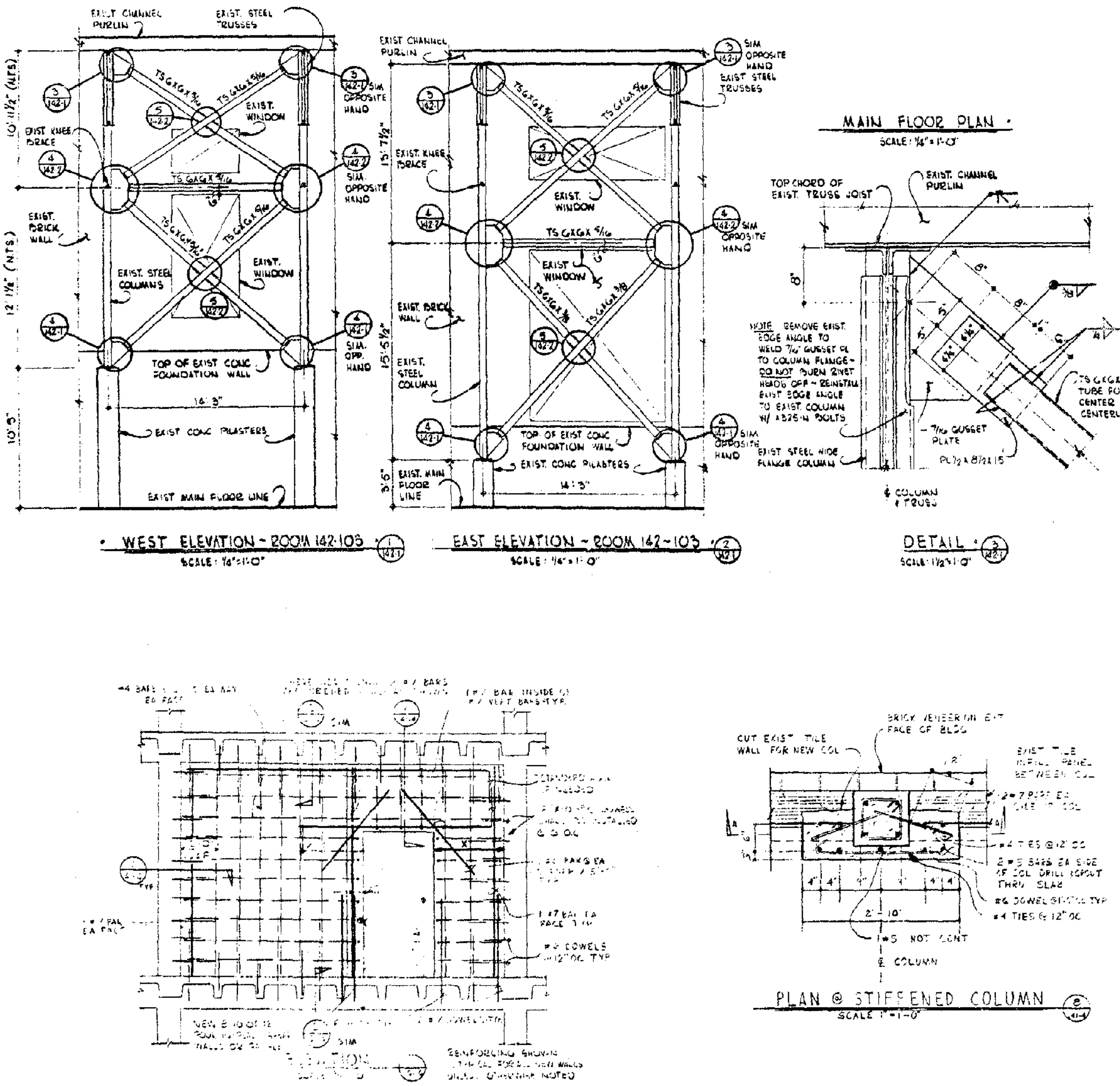
SALT LAKE CITY, UT

Figure 2



SALT LAKE CITY, UT

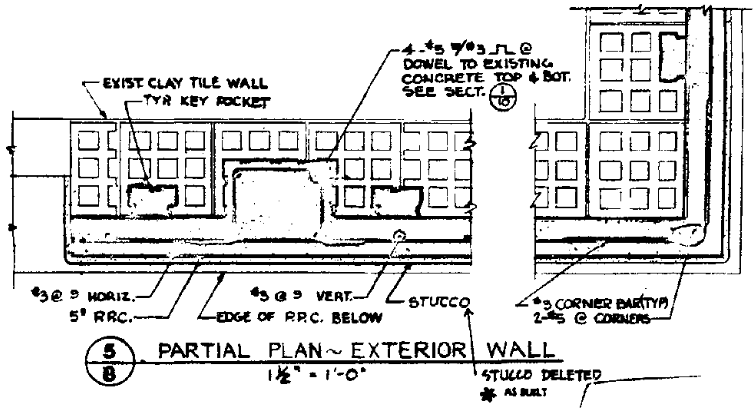
Figure 3

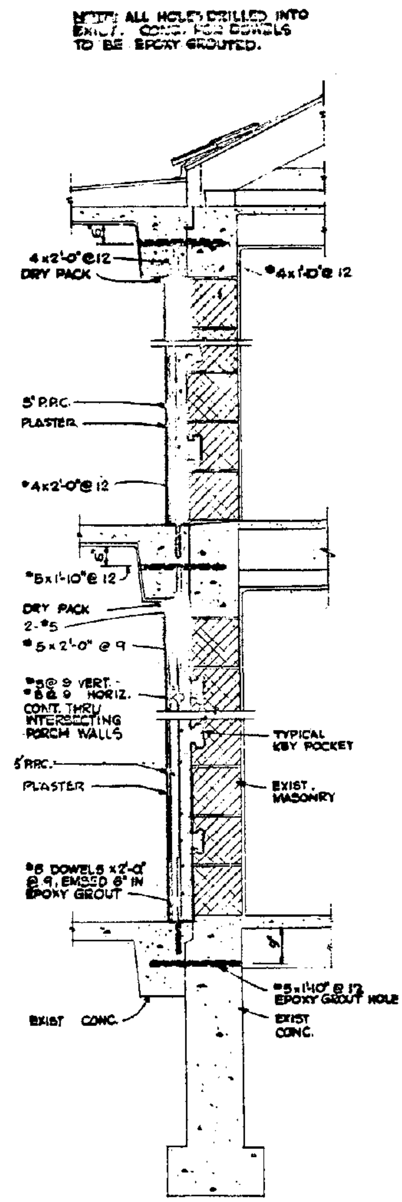


A/E: Grossman Whitney Griffin
Structural Consultant: URS/John A. Blume & Assoc.

FORT HARRISON, MT

Figure 4

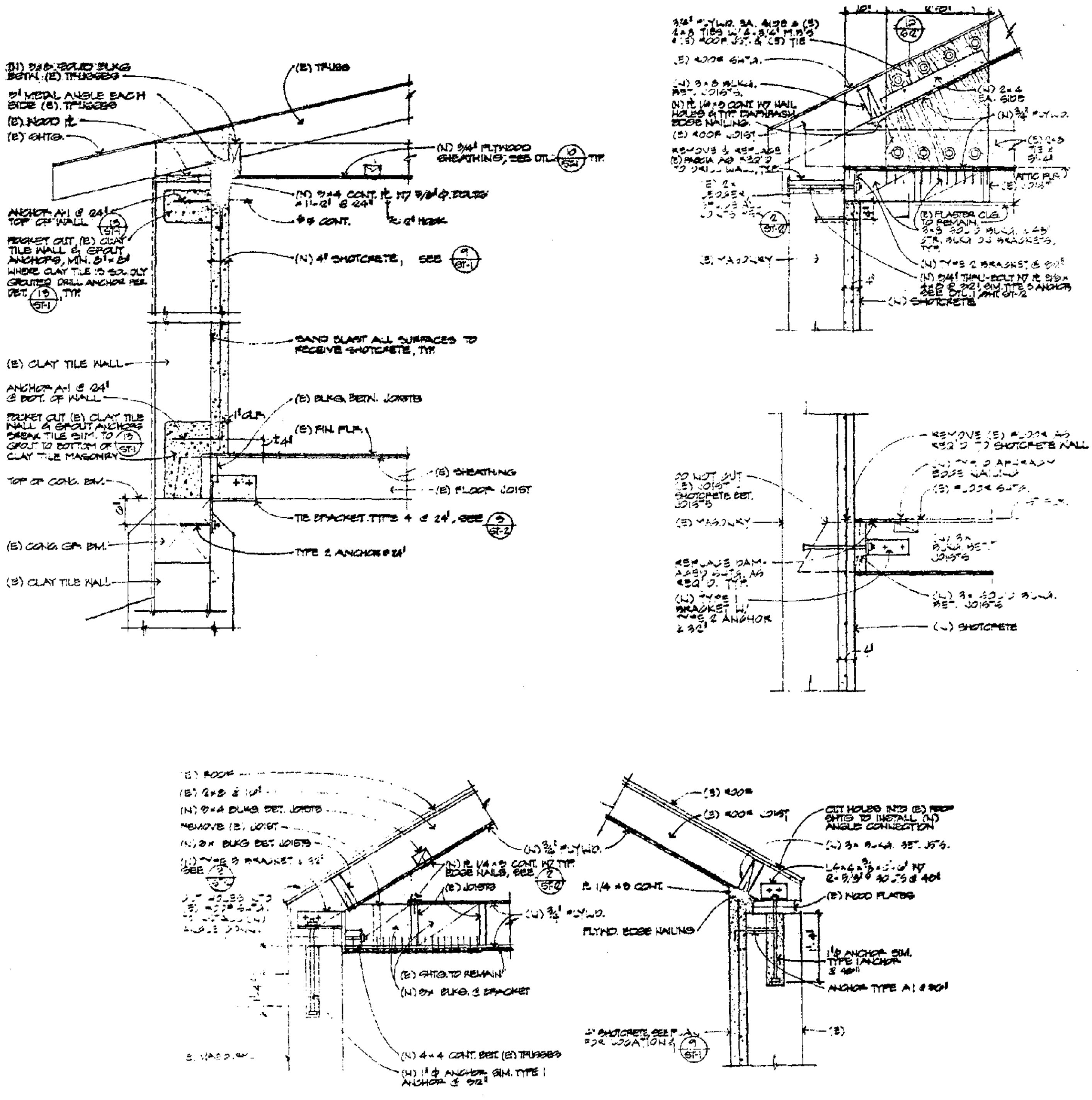




A/E: The Richardson Associates, Seattle, WA

AMERICAN LAKE, WA

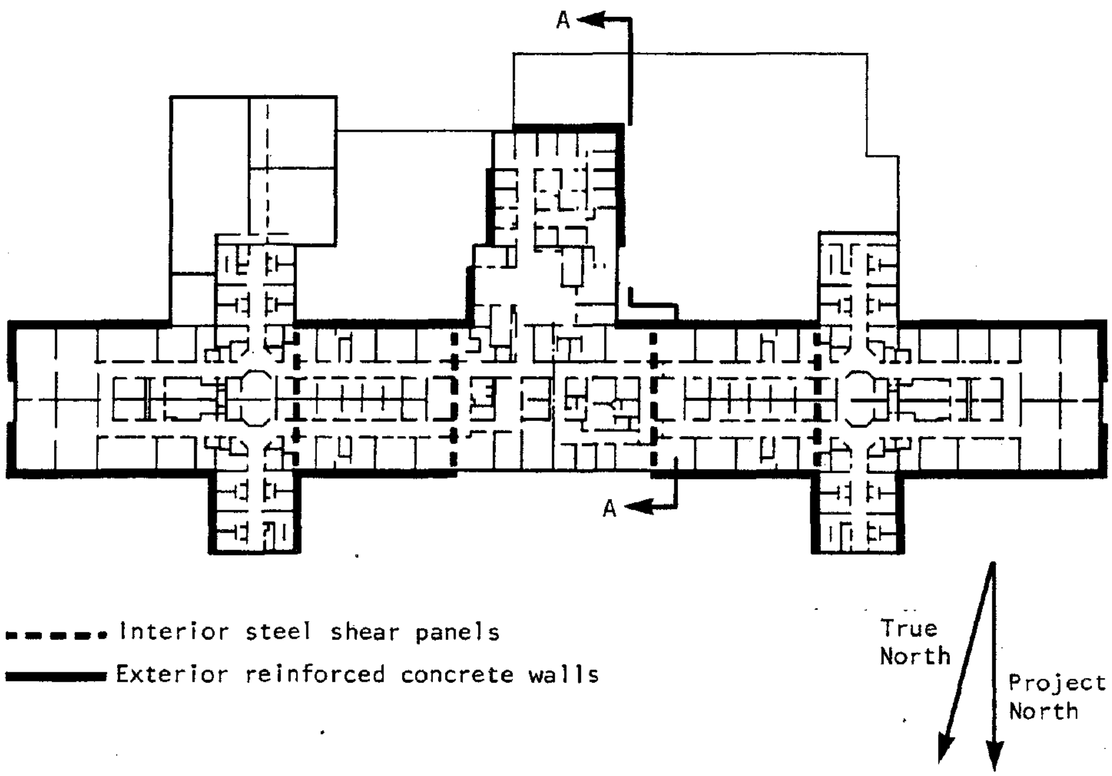
Figure 5



Structural Consultant: Holben & Martin, Tucson, AZ.

PRESCOTT, AZ

Figure 6

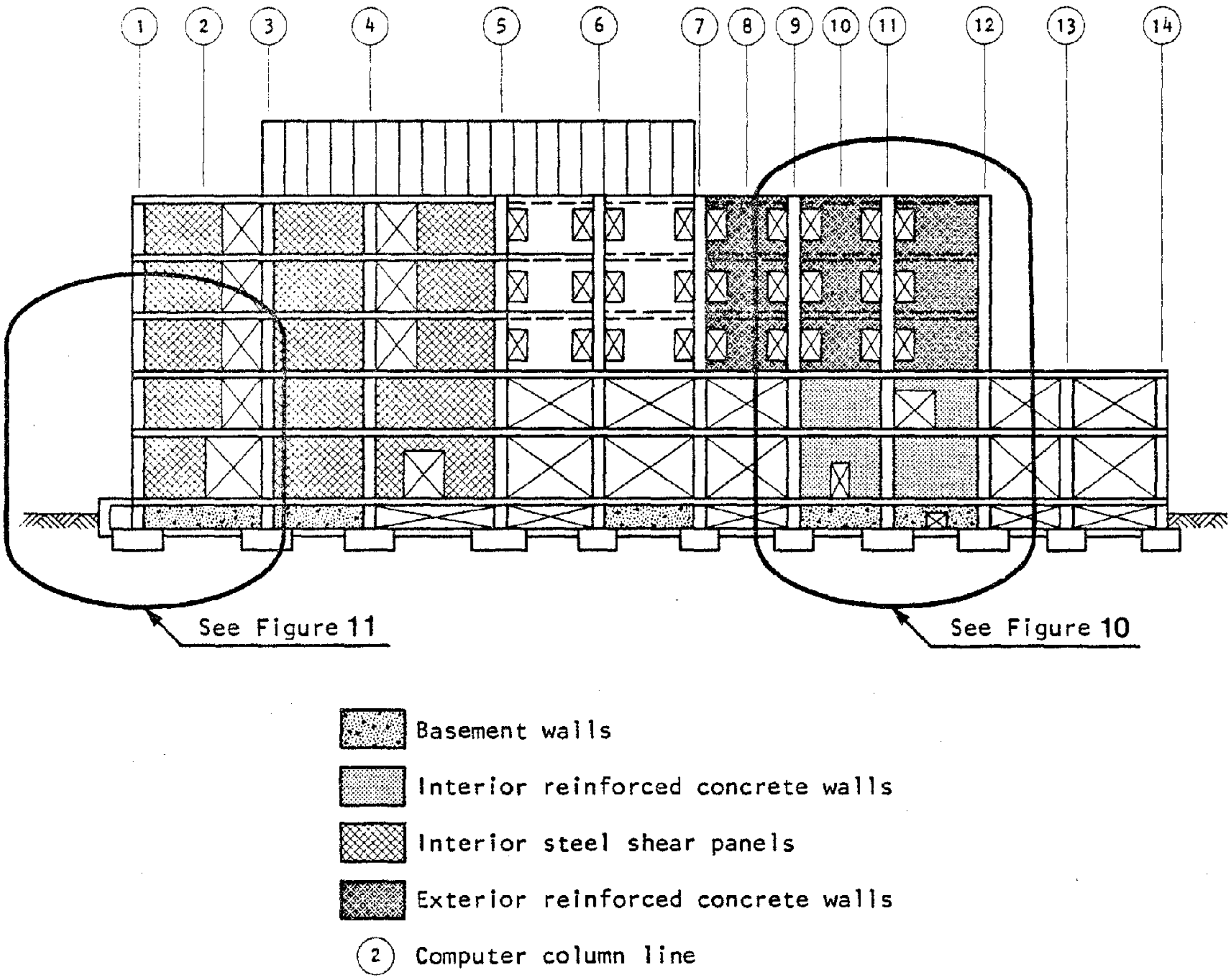


THIRD-FLOOR PLAN

A/E: Lafaye Associates
Structural Consultant: John A. Blume

CHARLESTON, SC

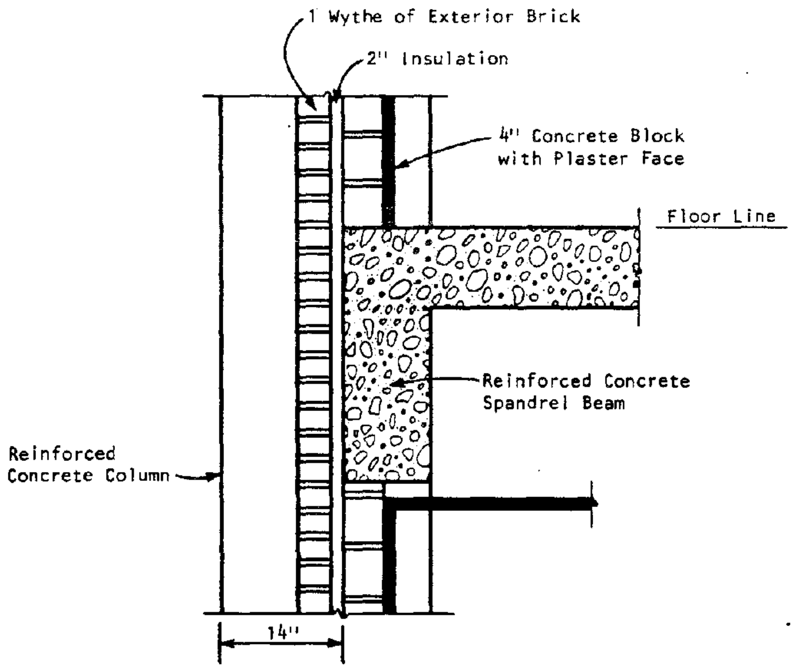
Figure 7



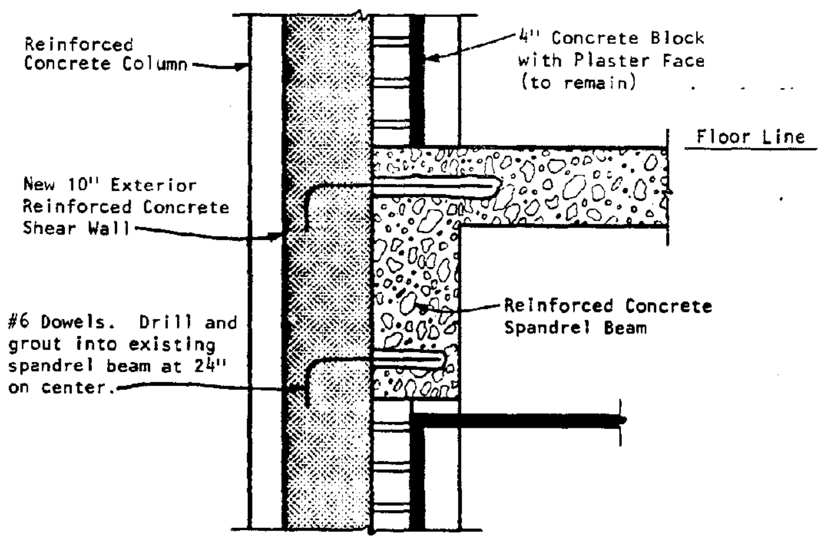
SECTION A, COMPUTER MODEL WITH NEW SHEAR WALLS

CHARLESTON, SC

Figure 8



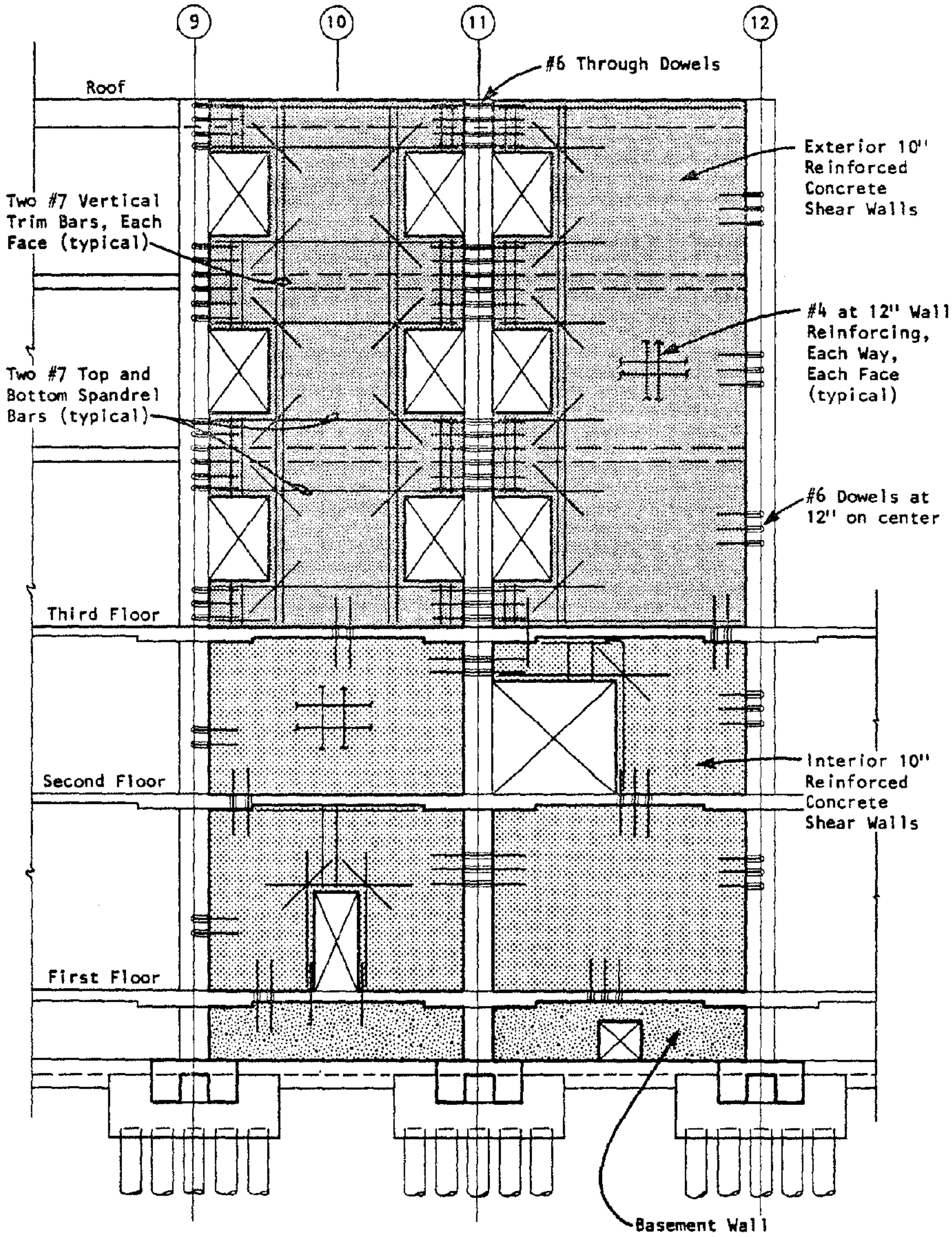
a. Existing Condition



b. Modified Condition

SECTION THROUGH EXTERIOR WALL

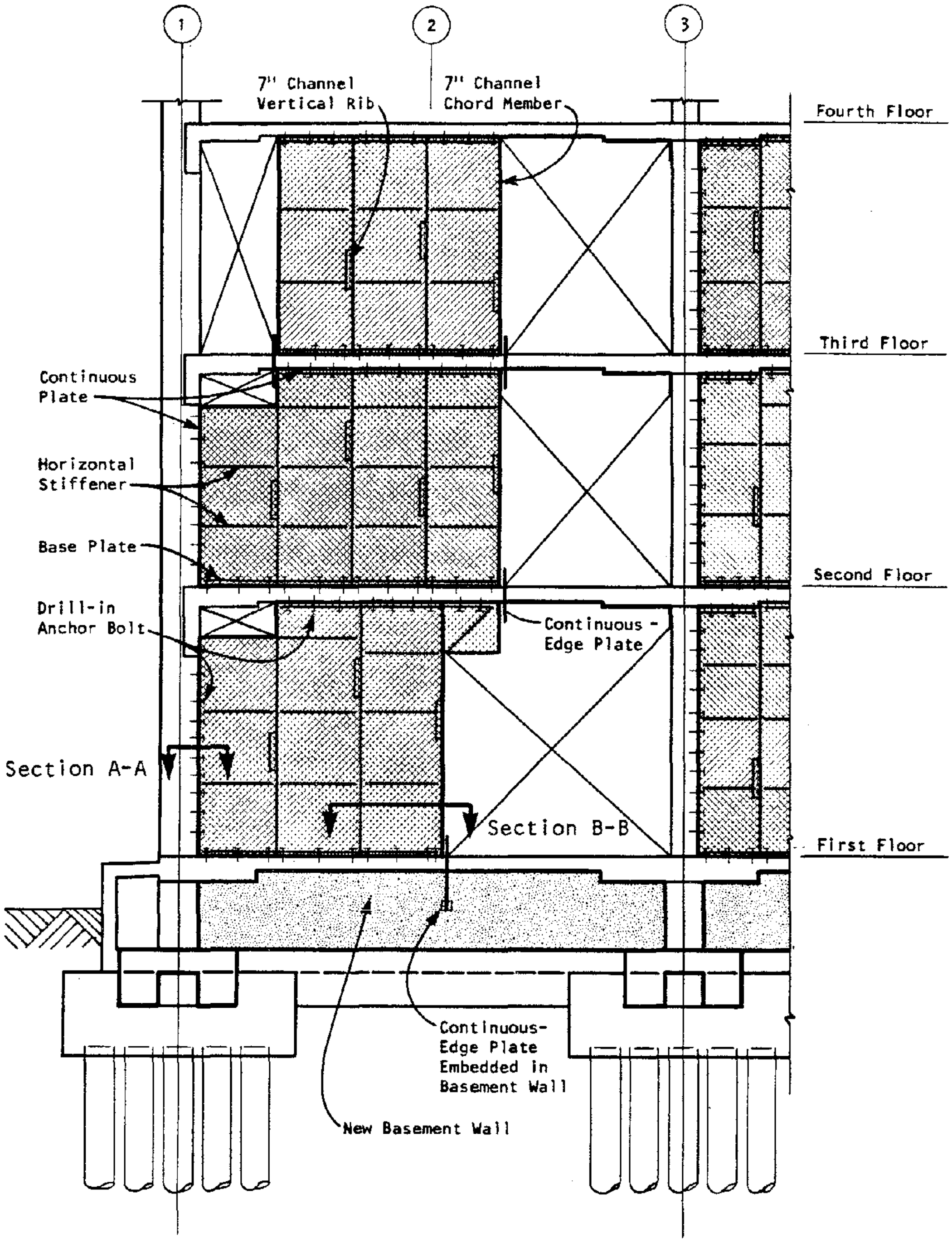
Figure 9 CHARLESTON, SC



WALL ELEVATION

CHARLESTON, SC

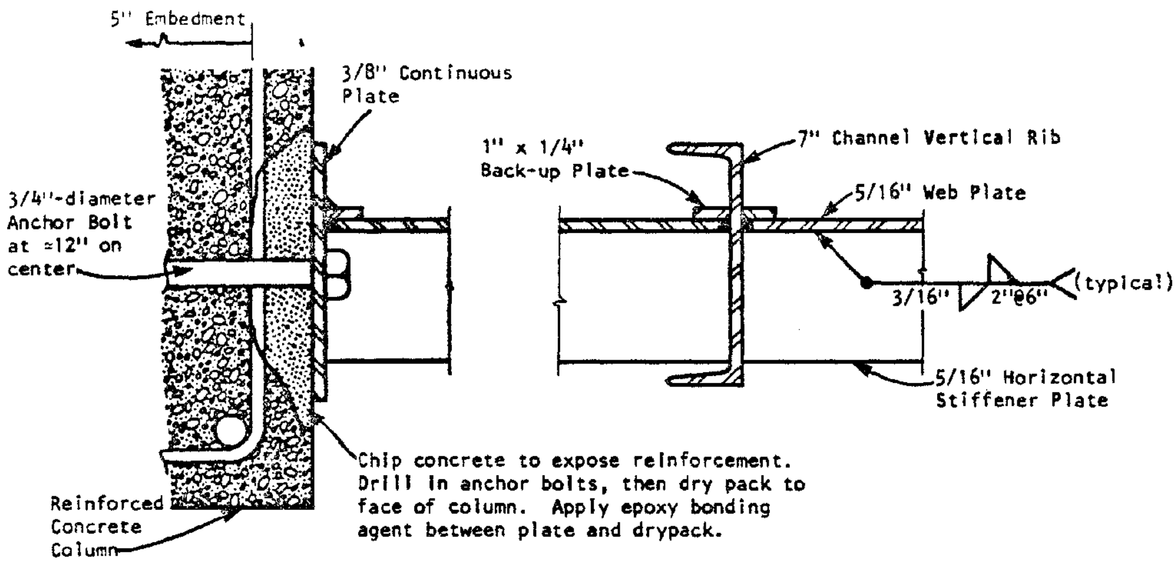
Figure 10



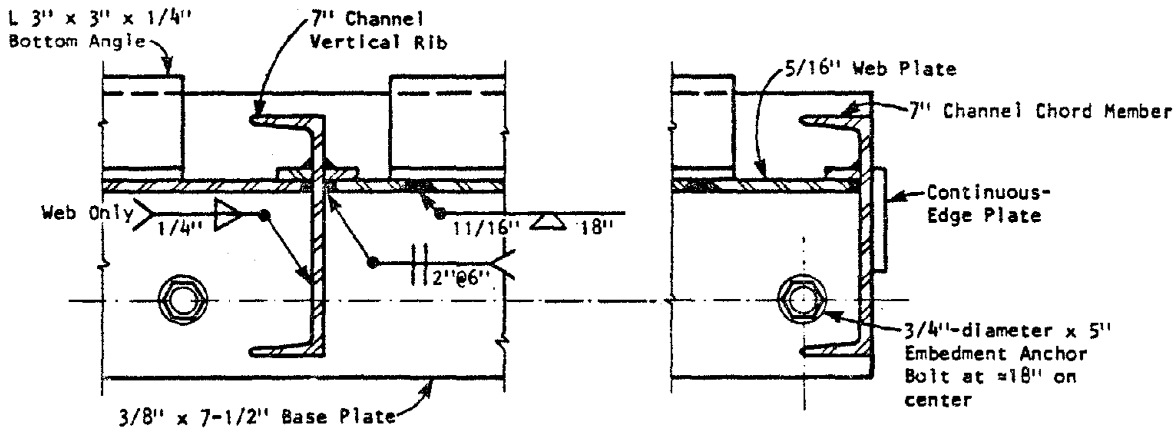
ELEVATION OF STEEL SHEAR PANELS

CHARLESTON, SC

Figure 11



a. Section A-A



b. Section B-B

STRUCTURAL DETAILS OF THE STEEL SHEAR PANELS

CHARLESTON, SC

Figure 12

EFFECTS OF INFILLS IN SEISMIC RESISTANT BUILDING

Vitelmo Bertero,[1] F. ASCE, and Steven Brokken[2]

INTRODUCTION

STATEMENT OF PROBLEM

Analysis of building performance during earthquakes has shown that numerous building failures have resulted because the building's basic bare resisting structural systems are designed neglecting the structural modifications introduced by the addition of infills. Recognition that the dynamic characteristics of the bare basic structural system are significantly changed by the incorporation of infills has led to the formulation of two building design philosophies in seismic resistant design. One philosophy requires that the infills be effectively isolated structurally from the structural system so that their structural effects can correctly be neglected. The second considers the infills to be tightly placed, and, therefore, their interaction with the structural system to resist the effect of all kinds of excitations should be properly considered in the design, detailing, and construction.

The first philosophy is conceptually attractive since it avoids the need to predict the interacting behavior of the infills, which at present offers great uncertainties. It is not surprising, therefore, that some countries have seismic codes which encourage the application of this philosophy, although the effective structural isolation of the infills presents serious practical problems, particularly in predicting and providing the required gap and in achieving effective flexible connection details. The authors believe that the second philosophy offers more conceptual and practical advantages, particularly if the basic structural system is moment resisting frame. This is because a main principle for seismic-resistant design is: "Avoid unnecessary masses," and, "If a mass is necessary, use it structurally to resist seismic

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(1) Professor of Civil Engineering, University of California, Berkeley

(2) Design Engineer, URS/John A. Blume & Associates, Engineers

effects"[3]. Thus if walls and partitions are needed and the economical material is masonry or concrete, attempts should be made to use these infills as structural elements. The proper use of infill elements can be of great practical value in strengthening and stiffening the usually very flexible moment resisting bare frame. The connection details between infill and frame are also simplified, but because of the interacting effects, the infills can be subjected to deformations and stress beyond their elastic resistance and produce brittle types of failure when masonry panels are used. This is not a serious disadvantage with respect to isolated panels, however, because it is recommended that the infill panels contain adequate reinforcement even in this case [10].

When the panel infills are tightly placed in the frame, the problem of avoiding premature failure raises the questions: (1) How should these panels be reinforced; and (2) How should they be connected to their surroundings? A comprehensive review of the literature available on these problems to 1974 [3] revealed the need for further research, and so an integrated experimental investigation was initiated in 1974 at the University of California, Berkeley.

OBJECTIVES AND SCOPE

The ultimate objective is to research the hysteretic behavior of infilled frames under actions similar to those caused by severe earthquake ground motions in order to obtain reliable data to formulate procedures for design, detailing, and construction. Integrated experimental and analytical work has concentrated on seismic-resistant buildirgs whose structural system consists of R/C frames infilled with masonry panels. Results obtained to 1978 have been reported in Refs. 2, 5, and 6. A second series of experiments on a 3-1/2 story and 1-1/2 bay subassemblage of an 11-story apartment building (shown in Fig. 1) have been recently completed. This subassemblage was built to one-third scale, instrumented and tested. A total of 18 tests were conducted to investigate the relative performance of various types of infilling materials and construction techniques. The effects of infills on the seismic resistant R/C construction was studied analytically and have been reported in detail in Ref. 4.

The authors believe that the results and techniques used to reinforce the infill and to connect the panel to the frame can be of interest to the profession, and, therefore, this paper is presented with the following objectives: (1) To summarize the experimental investigation and the results obtained; (2) To evaluate these results and to assess the practical use of infills in sites located in regions with differing seismic risk; (3) To formulate recommendations for the design of new seismic-resistant buildings with infilled frame structural systems, and for the retrofitting of existing buildings having R/C moment resisting frames as a structural system; and (4) To point out research needs.

DESCRIPTION OF EXPERIMENTAL INVESTIGATION AND RESULTS

SPECIMENS

Quasi-static cyclic load tests were performed on 18 specimens. The specimens were similar to those used in the first series of studies [5,6]. As illustrated in Figs. 1 and 2, these specimens consisted of 1/3-scale model subassemblages of the lower 3-1/2 stories of an 11-story, 3 bay-frame with infills in the 2 outer bays. The design of the prototype and model, as well as the construction of the specimen, are described in detail in Refs. 5 and 6. Four different types of infills were used. Two infills consisted of hollow-unit masonry: clay (Fig. 2(b)) and concrete block. The characteristics and construction of these infills are given in Refs. 5 and 6. The third type of masonry infills used consisted of split brick with exterior welded wire fabric (WWF) reinforcement (Fig. 3(a)). Split bricks were laid in mortar infilling the frame opening. Cross ties were left in the mortar bed as a provision for holding the welded wire fabric mat flat for subsequent construction stages and to basquet the bricks. The panel was allowed to sit undisturbed for at least 24 hours; two mats of welded wire fabric were then attached to it, one on each side, with care taken to tie the mat flat at a distance of 1/8 in. from the brick face using the cross tie already in position. The wires of the WWF mat were spliced to dowels left anchored in the confined regions of the bounding frame members (Fig. 3) so that the panel was firmly attached to the bounding frame. Bonding agent was then applied to both sides of the panel to assure good bonding between the mortar cover and brick. A mortar cover 5/8 in. thick was applied in two layers at each side of the masonry infill. The technique, based on pneumatically applied mortar, can be used advantageously to place the cover. The fourth type of infill was lightweight concrete panels.

REPAIR, STRENGTHENING AND RETROFITTING OF SPECIMENS

Repair Method. After an infilled frame loading program was completed, it was found that severe panel damage was generally confined to one level and so panel replacement was necessary at only one level. The damaged panel was removed with care taken to retain the reinforcing steel (or WWF) protruding from the frame which had been cast in place for panel reinforcement anchorage. Cracks in the beams and columns were repaired by epoxy injection. If crushing of concrete had occurred, all loose concrete was removed from the frame members leaving only sound concrete. The frame members were then reformed and the concrete recast. After the frame member forms were stripped, infilling proceeded with new panel reinforcement, lap spliced as required to the frame anchorage steel.

Strengthening Method. During some tests the spiral transverse steel was observed to fracture in critical inelastic regions of the columns in the first story, causing immediate brittle shear failure at that location in the column. Any type of repair became difficult and rendered this story level useless in subsequent testing. It was, therefore, decided to strengthen this story so that panels in other stories could be tested. Strengthening was achieved by placing a rather substantial amount of reinforcing steel in the panel opening and casting this story solid (5-in. thick) in concrete.

Retrofitting Method. To retrofit infill panels into an existing bare frame, this frame was drilled to attach an anchorage system for the panel reinforcement. This anchorage system consisted of steel plates attached to the beams with anchor bolts at 8 in. O.C. (200 mm) and to the columns with bolts at 4 in. O.C. (100 mm). Wedge anchors were used in the columns and the third-story beams. The first- and second-story beams were drilled completely through, threaded rods were inserted, and nuts were secured to plates on both sides of the beam to secure anchorage plates for welded wire fabric reinforcement anchorage (see Fig. 4).

TESTING OF SPECIMENS

The specimens were tested horizontally. Reference 6 discusses in detail the test set up. The models were loaded as shown in Fig. 2(a).

The ratio between the lateral force and corresponding overturning moment was calculated by a dynamic elastic analysis of the entire frame. Analyses were conducted on both the bare frame and the infilled frame. Overturning moment from stories above the subassemblage, as calculated from analysis, was applied automatically using a preset transfer between the horizontal and column jacks through a servocontrol system.

TEST RESULTS

During the first series of studies reported in Refs. 5 and 6 four tests were conducted. In this second series a total of 18 tests have been performed. The main results are summarized in Table 1 and some typical load-deformation relationships for the different types of specimens tested are illustrated in Figs. 5-10.

EVALUATION OF TEST RESULTS AND THE INPLICATIONS ON DESIGN AND RETROFITTING OF SEISMIC-RESISTANT BUILDINGS

INTRODUCTORY REMARKS

A detailed evaluation of test results obtained in the two series of tests conducted at Berkeley are given in Refs. 4 and 6. This evaluation, as well as analyses of results obtained by other researchers, have shown clearly that the infill significantly affects the stiffness (Table 2), strength (Table 1), damping, and deformation capacity (ductility) and consequently the energy absorption and dissipation capacities of the bare frame. All these effects result in changes in the dynamic characteristic of the building in which the infill is used, and, practically, the question is how the infill will affect the seismic response of the buildings and how these effects should be considered in design and retrofitting procedures.

This is not easy to answer, because the degree to which the infill affects the above mechanical characteristics of the structure depend upon: quality control of the infill materials, workmanship of the infill, and how the infill is reinforced and anchored or connected to the bare structure of the building. The infills not only modify the available (supplied) stiffness, strength (yielding and ultimate), damping, hysteretic behavior and deformation capacity of the building structure, but these changes also introduce modifications in the demands of these same response parameters to any given earthquake ground motion.

The addition of infills brings an increase in the building mass. This increase in mass has two main effects: (1) The reactive mass, M, is increased; and (2) The period, T, of the structure is increased. While the increase in reactive mass brings a direct increase in the inertia forces that will be developed for any given acceleration to which this mass will be subjected, the effect of a relative increase in the period T on the response of the structure depends on the interacting dynamic characteristics of the building and the ground motions. Furthermore, while the addition of the infills by virtue of its mass increases the period T, it also introduces an increase in stiffness and thus decreases the T. These opposite interacting effects and the changes in the effective viscous damping and in the mechanisms of dissipation of energy (because of changes in the overall pattern and amount of local inelastic deformations) make conclusions difficult regarding the final effects of the infill for general cases. However, an attempt to arrive at some conclusion for the particular case under consideration is believed worthwhile so as to provide trends to apply in the general case. Consequently, an evaluation of the effects of infills on most of the above parameters for the building considered in the Berkeley investigation is presented considering the two sides of the design equation, i.e., the effects on the demands as well as on the supplies.

EFFECTS OF INFILL ON THE SUPPLIED LATERAL STIFFNESS, K, AND ON THE PERIOD, T

The lateral stiffness of the subassemblage tested, based on the interstory drift, is given in Table 2. Because the initial tangential stiffness deteriorates very quickly at the service lateral load, an effective interstory lateral stiffness, K_I^S, at service load level has been evaluated and introduced.

In interpreting the significance of these values regarding the lateral stiffness of the prototype frame, K_I^p, it has to be considered that the interstory lateral stiffness of the model frame K_I^m can be considered as twice that measured in the tests of the subassemblage and that the K_I^p is equal to the K_I^m multiplied by the length scale L_s, i.e.,

$$K_I^p = K_I^m L_s = 2 K_I^s L_s . \tag{1}$$

This interstory lateral stiffness K_I^p will be used as representative of the lateral stiffness of the prototype.

Lateral Stiffness of Infilled Frames vs. Bare Frame. Comparing the values given in Table 2, it can be seen that considering average of K_I^s for infills of the same type, the smallest of all lateral stiffness of infilled frame, $(K_I^p)_{if}$, (obtained for the solid brick panels reinforced with welded wire fabric) was 4.66 times that of the bare frame $(K_I^p)_{bf}$. The largest of all the $(K_I^p)_{if}$ corresponding to the reinforced lightweight concrete was 10.94 times the $(K_I^p)_{bf}$ and in the average the $(K_I^p)_{if}$ was 6.31 times the $(K_I^p)_{bf}$.

Effect of $(K_I^p)_{if}$ on Period, T, of Building. Although in general the addition of an infill decreases the period, T, the specific amount of decrease depends upon how the total mass of the building, M, changes relative to the stiffness with the addition of infill. Depending on the assumption of how the M changes, the different results are summarized in Table 3, where two bounds regarding the changes in M have been evaluated. Upper bound, where all 11 frames of buildings of Fig. 1 are infilled, and lower bound, where only 4 of the 11 frames are infilled (considered to be the smallest desirable number of structurally stiffened frames [1]). For each of these two bounds two cases were considered, one in which the M is assumed the same as when the structure is considered as a bare frame, and the other in which the infill adds mass.

Analysis of the results obtained reveals that any of the infill, even the softest, will produce significant change in the T of the building. Furthermore, the effect of the added mass due to infills on the T, is very small and can be neglected.

Period of the Prototype Building, T^P. To have the values of T^P in secs for the prototype building, it is necessary to estimate its period where a bare frame structure building is used. This T^P can be analytically computed or estimated from the experimental results. The analytically computed value was 1.30 secs [6]. Using the experimental stiffness of the subassemblage and applying Eq. (1), considering as the prototype mass the estimated one of 23144 kips (102945 KN), the T^P_{bf} results to be equal to 1.01 secs. Using these two values as an estimation of the period of the bare frame building, it is possible to compute the period for the infilled frame building. These values are given in Table 3.

EFFECTS OF INFILL ON THE SUPPLIED STRENGTH TO THE BUILDING

These effects are again evaluated on the basis of the results obtained in the test specimens (model subassemblages), making different assumptions regarding the number of frames that are infilled in the real building. The evaluation of the strength is based on the estimation of the base shear strength, V_n, that the model of the building could have resisted. This estimation in turn will be based on the measured lateral resistance of the specimen tested, $(V_n)^s$, which is equal to the maximum lateral force H plotted in the diagrams of Figs. 5 through 10 and summarized in Table 1.

Base Shear Strength of Bare Frame, $(V_n)_{bf}$. Considering that the maximum measured lateral resistance H of the specimen in Test 15 was 12.5 kips (55.6 KN), the total lateral resistance of the model of the complete building, if the only resisting structural element were the 11 bare frames, would amount to 275 kips (1224. KN).

Base Shear Strength of Infilled Frames, $(V_n)_{if}$. As summarized in Table 1, the measured H varied from a minimum of 35.3 kips (157. KN) to a maximum of 100 kips (445. KN).

In evaluating the supplied strength to the prototype building or its model from the results obtained in the test of the model subassemblages, it is necessary to distinguish the two bounds considered previously, i.e., an upper bound based on the assumption that all 11 frames are infilled, and a lower bound assuming that only 4 of the 11 frames are infilled. The final results obtained from this evaluation are summarized in Table 4. When the 11 frames are infilled, the supplied lateral strength of the building, V_n, is directly proportional to the results obtained on the specimen tested, i.e.,

$$V_n = (V_n)^m L_s^2 = [(V_n)^s \times 2] L_s^2 \qquad (2)$$

When only 4 of the 11 frames are infilled, the determination of V_n requires analysis of the load-deformation relationship of the infilled frames, and that of the bare frame (Figs. 5-10), and an assumption regarding the in-plane flexibility of the floor system (diaphragm). To simplify the discussion, it will be assumed that the diaphragm is rigid and that no torsion is developed.

As illustrated in Fig. 11, the infilled frame reaches its peak "elastic" strength at a displacement (interstory drift) somewhat smaller than the one at which the bare frame reaches its maximum lateral strength. Thus the elastic strength of the building cannot be obtained adding the peak strength of the bare frame to that of the infilled frame. For each different type of infill it would be necessary to analyze the load-deformation of the infilled frame together with that of the bare frame. From inspection of the results obtained, it has been concluded that a lower bound of the strength can be obtained by considering that when the infilled frame reached its peak "elastic" strength, the bare frame had developed equal to half of its maximum strength, i.e., that the

$$[(V_n)^m_{bf}]_{(\delta_{if})_{max}} = 2 \times [1/2\ (V_n)^s_{bf}] = 12.5 \text{ kips } (55.6 \text{ KN}) .$$

As shown in Table 4, although the unreinforced masonry infill resulted in the lowest lateral resistance, it still was 2.82 times the resistance of the base frame when 11 frames were infilled, and 1.34 times when only 4 of the 11 frames were infilled. The largest increase in lateral resistance was obtained for the reinforced lightweight concrete infill, amounting to on the average, 672 and 212 percent increases, depending on whether 11 or only 4 of the frames were infilled.

ESTIMATION OF DEMANDS: EFFECTS OF CHANGE IN T

The dynamic response depends not only on the dynamic characteristics of the building (T, ξ, V_n and μ), but also on the dynamic characteristics of the ground motions. The easy way to obtain a clear idea of what the effects can be of the changes in T over the response, is to analyze the response spectra of the critical ground motions. In doing so the following two cases have to be distinguished: linear elastic and inelastic response. Before discussing these two cases, it is necessary to define the mass of the building, the period of the bare frame building, and to adopt an effective viscous damping ratio, ξ.

Mass, M, of the Building. Because the two main effects of the change in mass are small for this particular building, it will be assumed that the mass is the same 23144 $\frac{k}{g}$ (102990 $\frac{KN}{g}$) whether the structure of the building is considered as bare frame or infilled frame.

Period, T, of the Bare Frame Building. To illustrate how the initial stiffness of the bare frame can affect the influence of infills, the two following periods of the bare frame will be considered: the T^p_{bf} estimated from test results equals 1.01 secs, and the one obtained analytically, i.e., 1.30 secs.

Damping Ratio, ξ. Although the addition of infills may introduce considerable change in ξ, usually increasing it for large deformations, (values of $\xi = 12\%$ have been measured) for simplicity's sake, the ξ for the infilled frame building is assumed to be the same as for the bare frame building under strong ground motions, i.e., $\xi = 5\%$.

Linear Elastic Response. A linear elastic response spectra as suggested by Newmark and Hall [8], for a maximum effective peak acceleration of 0.5 g (Fig. 12), has been selected for discussion.

Effect of Changes in T on Seismic Force Demands, V^D_{if}. Table 5 summarizes this effect. Because of the decrease in T induced by the effect of the infills from 1.30 to 0.39 sec (in the case of the largest decrease), when all the frames are infilled the demands in design seismic forces increase about 141%. Figure 12 illustrates this increase. For simplicity it is assumed that the total seismic force demand is directly given by the first mode response, i.e., the response of the structure is considered as that of a single degree of freedom having the total mass M of the building and the periods computed in Table 4. In the case that $T^p_{bf} = 1.01$ sec, the addition of infills changes this value to 0.40, 0.46, and 0.30 secs for the average, lowest, and highest decreases. This change causes an increase in seismic force demands of 86%. Table 5 shows the estimated increase when only 4 of the 11 frames are infilled; the minimum increase is 56%. Increases in seismic forces of the order of 56% to 141% are very significant and cannot be neglected. It is clear that for the type of ground motions represented in the selected elastic response spectra, the more flexible the bare frame, the larger the increase in the seismic forces attracted by the addition of the infill.

Effect of Changes in T on Deformation Demands. Figure 12 illustrates how the maximum displacement decreases in 82% when the T_{bf} of 1.30 secs is reduced by the addition of the infill to a T_{if} of 0.39 secs. Table 6 summarizes the decrease in demands for all the different cases of infilled frames of the building considered above. It should be noted that even when only 4 frames are infilled, the decreases vary from 33% to 60%. These decreases in deformation are very significant and have beneficial effects: The smaller the deformation the smaller the damage, either to the structural or nonstructural components, and the smaller the P-Δ effects, which are two of the main drawbacks in the use of just bare moment resisting frame.

Overall Effect of Infills on Strength Demand and Strength Supply: Intensity of Motions that Infilled Frame Building Can Resist Elastically. Based on the results summarized in Tables 4 and 5, the following observations can be made regarding the overall effect of infill on strengths, when the behavior remains in the "elastic" range.

1. When all the bare frames of the building are infilled, the increase in supplied strength considerably exceeds the increase in strength demands.

2. In cases where only 4 of the 11 frames are infilled with panels having a $\rho \geq 0.4\%$, the increase in supplied strength is larger than the increase in demanded strength.

From the standpoint of "elastic" strength, it appears that the use of all types of infills (considered in the Berkeley investigation), when properly reinforced with $\rho \geq 0.4\%$, is advantageous, in comparison to the behavior of bare frame buildings. This is only correct, however, when it is possible to guarantee that the building will be able to supply the elastic strength demanded. Therefore, it remains to estimate what intensity of ground motions the supplied elastic strength will be capable of resisting. The main results of this estimation are summarized in Table 7. From comparison of results obtained between infilled frames and bare frame buildings, the following observations can be made.

(1) Case where all frames are infilled. Unreinforced masonry infills could be used advantageously (i.e., elastic strength supplied larger than elastic strength demands) in seismic regions in which the peak effective acceleration a_{ep} is ≤ 0.12 g, which, according to the ATC recommendations [1], is for most of the U.S. (areas 1, 2, and 3). In the case of reinforced lightweight concrete infills, these infills could be used without the danger of any significant damage in seismic regions in which $a_{ep} \leq 0.32$ g, which means they could be used in regions of very severe earthquake ground motions. The maximum value specified by ATC [1] for a_{ep} is 0.40 g.

(2) Case where only 4 of the 11 frames are infilled. Unreinforced masonry could be used in seismic regions where the $a_{ep} \leq 0.07$ g, i.e., in regions located in the U.S. area classified as 1 and 2 in the map area classification recommended by ATC [1]. The solid split bricks reinforced with welded wire fabric could be used advantageously with respect to bare frame in regions where $a_{ep} \leq 0.14$ g (i.e., for all 1, 2, and 3 areas according to ATC map area classification), without danger of suffering serious damage. Similarly, reinforced lightweight concrete infill could be used in areas where $a_{ep} \leq 0.17$ g, i.e., ATC areas 1 through 4.

It can be concluded that infilling moment resisting frames with properly reinforced panels offers advantages when designed so that the frames would remain in the elastic range during the most severe earthquake ground motion that can occur. But what would happen if these infills were subjected to deformations larger than those corresponding to its maximum "elastic" strength? Can the infilled frame survive such deformations without severe damage? In attempting to answer it is necessary to analyze the inelastic behavior of infills in the infilled frames, and how this behavior affects the performance of the frames.

Effect of Infill on the Inelastic Response of the Building. In the analysis of this effect it is convenient to distinguish the following cases:

1. *Ductile moment resisting frame infilled with unreinforced masonry.*

Under cyclic loading [Fig. 7b] as soon as the panel reaches its maximum strength (which occurs with very small amounts of inelastic deformations, approximately 1.5 times that which will correspond to linear elastic behavior, given a displacement ductility ratio, μ_δ, of about 2.5), there is a reduction in strength to a value that is close but somewhat higher (10%) than that observed in the experiments conducted with a first soft story frame (Specimen 9, Fig. 6) about 23 kips (102 KN), and then an increase up to a value of about 30 kips (133 KN) up to a μ_δ of about 39. It should be noted that after a μ_δ of 2.5, some portions of the unreinforced infill started to spall out. If an analysis using inelastic response spectra similar to those shown in Fig. 13, but for $\mu_\delta = 2.5$ is conducted, the increase in strength demand due to the decrease in T from 1.30 secs to 0.52 secs is found to be 138%, while the experiments show that the increase in the supplied strength is 182% for μ_δ up to 2.5. Therefore, regarding strength, it appears that ductile moment resistant frame with unreinforced infills can be used advantageously in regions where a_{ep} is ≤ 0.26 g if all the 11 frames are infilled, or $a_{ep} \leq 0.22$ g if only 4 of the 11 frames are infilled. The real problem with this kind of infill is not initial stiffness or strength, but that with panels having large dimensions, as those under study, as soon as maximum strength is reached the masonry units can shatter and large portions of the infill spall out. In earthquake response, this is like an explosive failure with shedding of large portions of unreinforced masonry all around. This type of explosive failure with shedding of large portions of unreinforced masonry all around. This type of explosive failure of unreinforced masonry infills has been typically observed after moderate to severe earthquake ground motion. In general it is inadvisable to use unreinforced masonry infills except in cases where the response demands will not exceed the elastic range, and where out-of-plane failure of the infills can be restrained.

2. *Nonductile moment resisting frame infilled with unreinforced masonry.* This case is similar to the previous one but even more dangerous because the

explosive type of failure of the infill leads the infilled frame to behave like one soft story frame with very large demands in shear and plastic rotations in the columns and/or the beams or beam-column joints adjacent to the failed infilled panel. As these elements have not been designed to resist such demands, the explosive failure of the unreinforced masonry usually will lead to the collapse of the frame. Thus this system should not be used except for cases where the building can resist elastically the effect of the most severe earthquake ground motion. Therefore its use should be limited to regions of very low seismic risk level, i.e., regions where $a_{ep} \leq 0.12$ g if all the frames are infilled or $a_{ep} \leq 0.07$ g if only 4 of the 11 frames are infilled.

3. *Properly designed ductile moment resistant frame infilled with reinforced masonry or concrete panels.*

(1) Reinforced masonry infills. Experiments conducted on these types of infills show that maximum strength is reached at a deformation which can be considered as being at least two times the deformation which would result if a linear elastic behavior with an initial effective stiffness occurs. Thus it can be assumed that μ_δ at the average peak strength of the reinforced masonry infill, $(V_n)^m_{rif}$, is at least equal to 2. Therefore, the reinforced masonry infilled frame building on the average can resist seismic ground motions (of the types given a design response spectra as that of Figs. 12 and 13) having the following peak accelerations: If all 11 frames are infilled $a_{ep} = 0.40$ g for T = 0.52 secs and $a_{ep} = 0.38$ g for T = 0.40 secs; If 4 of the 11 frames are infilled $a_{ep} = 0.26$ g for T = 0.75 secs and $a_{ep} = 0.18$ g for T = 0.54 secs.

In the case where the infill consisted of solid split bricks reinforced with two layers of WWF--since the infilled frame can develop a $\mu_\delta = 4.2$ with a reduction of only 14% in strength (Figs. 10 and 11), it becomes evident that this type of structural system can resist earthquake ground motions having the following a_{ep}: If all 11 frames are infilled $a_{ep} = 0.77$ g for T = 0.60 secs and $a_{ep} = 0.59$ g for T = 0.46 secs; If 4 of the 11 frames are infilled $a_{ep} = 0.55$ g for T = 0.84 secs and a_{ep} = 0.44 g for T = 0.66 secs.

In the case of a building with bare ductile frame--for a T_{bf} = 1.30 secs it would require developing a $\mu_\delta \geq 6.1$ to be able to resist a ground motion with an a_{ep} = 0.55 g, and for a T_{bf} = 1.01 secs it would require a $\mu_\delta \geq 5.6$ to resist an a_{ep} = 0.44 g. Since experiments have shown that the bare frame structure can develop a $\mu_\delta = 6.1$ without any significant loss in strength, it would appear that there is no advantage in using infills except when the majority of the frames are infilled. However, it should be recognized that for a bare frame structure to develop a $\mu_\delta = 6.1$, it would have to undergo lateral displacements considerably larger than that needed for an infilled frame building to develop $\mu_\delta = 4.2$. Furthermore, while in the case of the infilled frame, most of the damage will be developed in just one or two stories where the inelastic deformations are concentrated; in the case of the bare ductile moment resisting frame, the damage will spread throughout the whole height.

In the case of solid split bricks reinforced with WWF, the specimens were deflected, producing an interstory drift of 2.4 in. at the story where inelastic deformation was concentrated. This drift, which means an interstory drift ratio of 0.07, was achieved without any significant spalling of debris. This interstory drift, when translated in ductility displacement, means a $\mu_\delta \doteq 14$ which was attained with a reduction of strength of 32 percent (see Fig. 11). Therefore, this specimen could resist the following a_{ep} without danger of failure (collapse): <u>If all 11 frames are infilled</u> a_{ep} = 2.05 g for T = 0.60 secs and a_{ep} = 1.54 g for T = 0.46 secs; <u>If 4 of 11 frames are infilled</u> a_{ep} = 1.62 g for T = 0.84 secs and a_{ep} = 1.31 g for T = 0.66 secs.

The interstory drift ratio of 0.07 is very large, demanding large rotations in the columns. The columns of the specimen were capable of developing these rotations because of their special design and detailing. Actually the columns were capable of inducing an interstory drift index of 0.12 without losing flexural strength. Nonductile R/C columns cannot develop the plastic rotations required to obtain such an interstory drift ratio. Note, if an 11-story frame develops a complete collapse mechanism through plastic hinges at the beams, the interstory drift required to achieve the same displacement as the one with a soft story requiring an interstory drift index of 0.07, would be approximately 0.7/11 = 0.006.

In conclusion it can be stated that the use of specially designed moment resistant frame infilled with reinforced masonry, particularly solid split bricks with W.W.F., can be used advantageously for even the most severe seismic regions of the U.S.; provided the number of stories is limited to, say 11. This limitation is necessary because the inelastic deformation in this type of structure is usually concentrated in one or two stories, the larger this number of stories of a building the larger will be the demand in the story in which this inelastic deformation is concentrated. Furthermore, the frame has to have very ductile members because the inelastic demands at the story in which the inelastic deformations concentrate, would be very large. This problem has been discussed by Park and Paulay [9], who show that the required column curvature ductility factor ϕ_{uci}/ϕ_{yci}, can be typically expressed as $\phi_{uci}/\phi_{yci} = 12.54\ r - 3.2$ where r is the number of the story to the top of which the deflections are to be measured.

(2) Reinforced lightweight concrete infills. This type of infilled frame is capable of dissipating energy with a ductility somewhat larger than 2 without any loss in strength. However, for a μ_δ just larger than 3 the strength reduces rapidly to a value somewhat higher than the strength corresponding to the soft story frame. Considering a $\mu_\delta = 2$, it has been estimated that buildings with this type of infilled frame can resist ground motions with the following a_{ep}: If all 11 frames are infilled $a_{ep} \leq 0.54$ g for T = 0.39 secs and $a_{ep} \leq$ 0;54 g for T = 0.30 secs; If 4 of 11 frames are infilled $a_{ep} \leq 0.31$ g for T = 0.61 secs and $a_{ep} \leq 0.25$ g for T = 0.47 secs. Although there is a significant reduction in lateral strength after reaching its maximum value, the filure is far from being sudden or brittle (Fig. 9). For example, for a reduction in strength of 24.5%, the $\mu_\delta \doteq 4.3$. Consideration of these values leads to the following

estimated a_{ep} for the prototype buildings: <u>If all 11 frames are infilled</u> $a_{ep} \leq 0.68$ g for T = 0.39 secs and $a_{ep} \leq 0.64$ g for T = 0.30 secs; <u>If 4 of the 11 frames are infilled</u> $a_{ep} = 0.53$ g for T = 0.61 secs and $a_{ep} = 0.41$ g for T = 0.47 secs. Considering the value at which strength appears to be stabilized, 42 kips (187 KN), which is considerably higher than the 27.4 kips (122 KN) which is the maximum lateral resistance of a bare frame soft story, and that the inelastic deformation at this level gives a $\mu_\delta = 6.6$, the following values of a_{ep} can be obtained: <u>If all 11 frames are infilled</u> $a_{ep} \leq 0.64$ g for T = 0.39 secs and $a_{ep} \leq 0.48$ g for T = 0.30 secs; <u>If 4 of the 11 frames are infilled</u> $a_{ep} \leq 0.37$ g for T = 0.61 secs and $a_{ep} \leq 0.28$ g for T = 0.47 secs.

From analysis of the above results it can be concluded that R/C bare frame buildings of the type investigated can be advantageously infilled with reinforced lightweight concrete for even the most severe seismic regions of the U.S. if all the frames are infilled, and for the ATC map areas 1, 2, 3, 4, and 5 if only 4 of the 11 frames are infilled.

4. *<u>Nonductile Moment Resistant Frame Infilled with Reinforced Panels</u>*. In general this type of construction is not advisable if significant inelastic deformation is expected. In infilled frames the inelastic deformation is concentrated within a few stories, usually the lower ones, so ductility demands on the frame members of these stories can be very large, consequently these members should be ductile. Because of this type of behavior a designer could be tempted to design as ductile only the members of the story or stories in which inelastic behavior of the infill is expected. To design in this manner appears logical and economical, however, the designer must be aware that the results obtained in this investigation, as well as in others, clearly show that for such a design to work it must be assured that the inelastic deformation will actually concentrate in the weakest spot, i.e., the story that is designed as ductile. This is not an easy task. The uncertainties involved in predicting the critical

seismic response of buildings are so large that conservative precautions should always be taken. Furthermore, the strength, stiffness and deformation capacity of masonry infills are very sensitive to quality control of the materials and workmanship. To believe that it is possible to control "exactly" where inelastic deformations can occur in a real building is too optimistic.

CONCLUDING REMARKS REGARDING THE USE OF INFILLS IN THE SEISMIC-RESISTANT DESIGN AND RETROFITTING OF BUILDINGS

The evaluation of results and the observations made above were assessed regarding their implications to the design of new buildings and retrofitting of existing ones. The implications arrived at are stated under conclusions. A more detailed discussion is given in Ref. 4. It should be emphasized that most of the evaluations have been made through approximate numerical analysis which have been conducted to obtain trends or guidelines and not to represent or to obtain accurate predictions of actual behavior. Therefore, while the specific values may be questioned, it is believed that the trends and guidelines, and subsequently the conclusions given below, are valid.

CONCLUSIONS AND RECOMMENDATIONS

INTRODUCTORY REMARKS

Several observations and conclusions have been formulated in evaluating the experimental results and the effects of infills in the design and retrofitting of seismic resistant buildings whose structural systems are based on moment resisting space frames. In view of the relatively small amount of experimental data on which these conclusions are based, and the idealizations, simplifications, and assumptions made in the numerical analysis conducted, it is convenient to clearly recognize the constraints surrounding the validity of the conclusions so that they will not be misused. These limitations are summarized regarding the following parameters:

1. Type of Frame. A specially designed R/C moment resisting space frame of 3 bays and 11 stories.

2. Type of Infills. Unreinforced and reinforced masonry units (hollow and solid bricks, and concrete blocks) and lightweight reinforced concrete.

3. Quality Control of Materials. Although the masonry units used in construction were carefully selected and the grout, mortar, and concrete carefully designed, mixed, placed, and cured, considerable variations in the mechanical characteristics of these materials were observed. The results indicated that the behavior of the infill is very sensitive to variations in the quality of material and, therefore, good quality control of all material is a must for infills, particularly masonry infills.

4. Workmanship. Some weaker, stiffer, and premature types of inelastic behavior and pattern of cracking and/or crushing were attributed to lack of uniform workmanship in laying the masonry units and in the anchorage of the infill to the frame; thus excellent workmanship is required.

5. Infill Panel Arrangement. The two external bays of the 3 bay frames were fully infilled, i.e., without any opening, and formed what could be called a "coupled infilled frame."

6. Type of Building Considered in the Assessment of the Implications of Results Obtained. Regular buildings having a rectangular plan consisting of 11 frames of 3 bays and of 11 stories high where the frames are fully infilled, as described in item 5, and the locations of these infilled frames are such that no significant torsional forces are induced during the seismic response of the building. The importance of this limitation cannot be overemphasized.

7. Idealization of the Actual Lateral Load-Deformation Relationships of the Bare and Infilled Frames. The analytical assessment of the implications of the experimental results regarding behavior of the building have been made idealizing the actual experimental relationship by a linear elastic-perfectly plastic model using different yielding strengths and ductility levels.

8. Dynamic Characteristics of Building Site and of Ground Motions. It is assumed that the building is on firm ground and a "rigid foundation" can be constructed, and that all the ground motions that can occur have dynamic characteristics similar to those included in the derivation of the smoothed linear elastic and inelastic design response spectra suggested by Newmark and Hall [8] and illustrated in Figs. 12 and 13. The importance of the limitations imposed by these assumptions in conjunction with the idealization pointed out it item 7 should be emphasized, particularly where significant inelastic behavior is involved in the response. The effects of ground motions containing severe acceleration pulses (high a_{ep}) of long duration should be investigated before the conclusions from these results are applied to the design of new buildings and/or to retrofitting of existing buildings. The interacting effects of the observed significant deformation softening after reaching peak lateral resistance, with long acceleration pulses input, can lead to deformation demands considerably higher than those predicted by a linear elasti-perfectly plastic idealization. [7]

9. Reliability of the Analytical Results. In view of all the assumptions, idealizations, and uncertainties involved in the conducted analyses, the numerical values obtained should be considered as approximate and indicating trends, rather than an exact representation of what can be expected in specific cases.

CONCLUSIONS

Conclusions Regarding Overall Behavior of the Infilled Specimen Tested.

1. The addition of either unreinforced or reinforced infill to moment resisting frame increases significantly the lateral stiffness and lateral resistance of the frame.

2. As soon as cracking occurs, which happens very early, at service lateral load level, the initial tangential lateral stiffness decreases significantly, up to 80 percent, to a value that remains practically constant for a long range of

lateral load. To represent this behavior an effective interstory stiffness at lateral service load has been defined.

3. The lateral stiffness and strength depends on the history of loading. Under monotonically increasing load these two characteristics depend on the type of infill, the highest being for the lightweight concrete and the lowest for the brick. These characteristics do not depend upon how the panel is reinforced but they are sensitive to the quality control of the materials and to how well the infill is made, particularly to the workmanship along the interfaces of the infills and the boundary frame elements.

4. Hysteretic behavior depends upon the type of infill, the amount and arrangement of reinforcement, and the way that the panel is attached (anchored) to the frame. The cyclic loading of unreinforced infills leads to considerable deterioration in stiffness and strength when compared with the values observed under monotonic loading. This deterioration is due to propagation of infill damage that usually concentrates in one story. The peak strength under cyclic loading, which is smaller than that obtained under monotonically increasing load, deteriorates as the severity of deformation and number of cycles increases, but remains somewhat larger than the strength of a frame with a soft story corresponding to the story in which damage of the infill concentrates. Excellent hysteretic behavior has been obtained with the use of solid brick masonry infills externally reinforced with welded wire fabric covered with cement mortar.

5. Although the interstory displacement ductility under peak strength is small, about 2, large values are obtained under reduced strength. In the case of solid brick externally reinforced with welded wire fabric, this ductility was 4.2 under 86% of the peak strength, and reached the value of 14 under 68% of peak strength.

6. Except for very few specimens (Specimen 18 and one reported in Ref. 6) whose failure mechanisms involved two stories, in all other specimens the damage

concentrates in one story, consequently the final mechanism of failure is what can be defined as "a somewhat strengthened soft story frame." Thus the energy dissipated by an infilled R/C frame should be larger than a bare soft story frame.

7. Failure of unreinforced masonry infills was accompanied by production of substantial debris containing hazardously large pieces of masonry. The amount of debris in reinforced infills was smaller and most was contained in the plane of the infill, particularly in solid brick masonry reinforced externally with welded wire fabric.

8. The effective viscous damping coefficient of the virgin specimens is smaller than 2 percent. As soon as cracking develops the value of this damping coefficient increases up to 12 percent.

Conclusions from Comparison of Behaviors of Infilled Frames and Bare Frame

1. The initial tangential interstory lateral stiffness of the virgin infilled frames was more than 10 times the similar stiffness of the bare frame.

2. The effective interstory lateral stiffness of virgin infilled frames was 5.3 to 11.7 times the lateral stiffness of the bare frame depending on the type of infill, the smallest being for the clay brick and the largest for the lightweight concrete infill.

3. In case of repaired infills and retrofitting of repaired frames, the effective interstory lateral stiffness of the infilled frame was at least 3.4 times that of the virgin bare frame.

4. The maximum lateral resistance of virgin infilled frames was 4.8 to 5.8 times that obtained for the bare frame. For cases of repaired infills and retrofitting of repaired frames the maximum lateral resistance was 2.8 to 8.0 times that of the bare frame. The maximum increase has been obtained with lightweight concrete infills and the minimum with clay bricks.

5. The interstory displacement ductility ratio of the infilled frame is smaller than that of a bare frame but larger than that of a bare soft story

frame. For what can be considered a maximum acceptable interstory drift index, say 0.02 or even for values of this index up to 0.07, the hysteretic behavior of the solid brick masonry externally reinforced with welded wire fabric was superior (large energy absorption and energy dissipation capacities) to that of the bare frame.

6. The addition of infills introduces significant changes in the dynamic characteristic of the bare moment resisting frame. It modifies significantly the periods, modes of vibration, and the damping of the specimens. In the linear elastic range the fundamental period is decreased more than 54%, while the mass is increased in not more than 10%. The effective viscous damping coefficient is increased considerably, up to 500%. In the inelastic range the pattern of lateral deformations changed fundamentally because most of the significant inelastic deformations concentrate in one, or at the most, two stories.

<u>Conclusions Drawn from Assessment of the Implication of Experimental Results Obtained Regarding the Seismic Resistant Design of Buildings</u>

1. The addition of infill into the moment resisting frames of a building introduces significant changes in the dynamic characteristics of the building which should be considered in its design. These changes depend upon the number of frames that are infilled as well as the location of these frames.

2. The mass is increased, however, even when all the transverse frames of the building under consideration (Fig. 1) are infilled; the increase with respect to a bare frame building is only about 10%. This increase in mass has two main effects. First, it induces a change in the period of the building which is about 5%, therefore, it can be considered negligible in front of the uncertainties which exist in estimating the values of other main parameters. Secondly, the increase in mass increases directly the reactive mass, in 10% at the most, thus it increases the inertia forces that are developed during the seismic response.

3. The stiffness of the building is increased significantly in the case where all the frames are infilled, the increase varies from

366% to 994%. If only four of the frames are infilled the increase varies from 136% to 353%.

4. If the 11 frames are infilled the decreases in the fundamental period varies from 54% to 70%. If only four frames are infilled, the decrease varies from 35% to 53%.

5. The value of the effective viscous damping ratio for the whole building increases when compared with a bare frame structure and, therefore, will result in a decrease in its seismic response.

6. Strength Supply. Addition of infills to the frames increases the available (supplied) strength of the bare frame building significantly. If all the 11 frames are infilled the lateral strength in the transverse direction of the building is increased in 182% up to 700%, depending upon the type of infills. In the case where only 4 of the 11 frames are infilled, the increase varies from 34% to 255%. The smallest increase corresponds to the unreinforced masonry infills and the largest is produced by the reinforced lightweight concrete.

7. Strength Demands. For linear elastic behavior the addition of infills to the bare frame increases the strength demands in 86% up to 141% when all the frames are infilled, and in 56% to 141% when only 4 of the 11 frames are infilled.

8. Supplied Strength vs. Demanded Strength in the Case of Elastic Behavior. From comparison of values given in the above conclusions 6 and 7, it can be concluded that, except for cases of unreinforced infills in which only 4 of the 11 frames are infilled, the increase in supplied strength is larger than the increase in the demanded strength, thus from the viewpoint of strength it is beneficial to add infills.

9. Deformation Demands in the Case of Elastic Behavior. The addition of the infills decreases the demands on maximum displacement with respect to that corresponding to the bare frame building. The decreases vary from 56% to 85% in cases where all the frames are infilled, and 33% to 60% in cases where only 4 of the 11 frames are infilled. This decrease in displacement demand is a

significant advantage in the use of infills.

10. From conclusions 8 and 9 it is obvious that if it is possible to design the building to remain in the "elastic" range, then it is advantageous to add any of the types of infills, even unreinforced masonry, if all the frames are infilled and $a_{ep} \leq 0.12$ g. In cases where only 4 of the 11 frames are infilled, it is advantageous to add any type of infills reinforced with $\rho \geq 0.4\%$ that have been considered in this study. While a bare frame building can resist elastically ground motions similar to those considered in the derivation of the response spectra of Fig. 12 with an effective peak acceleration of $a_{ep} = 0.10$ g, the addition of infills of solid bricks reinforced externally with wire welded fabric allows the building to resist an $a_{ep} = 0.21$ g, i.e., an increase in 110% in intensity of ground motions if all the frames are infilled. If only 4 of the 11 frames are infilled it can resist an $a_{ep} = 0.14$ g, i.e., an increase of 40%. By infilling all the frames with reinforced lightweight concrete it is possible to resist elastically ground motions with an $a_{ep} \leq 0.32$ g, which means that they can be used in all the seismic regions of the U.S. except those classified as area 7 in the ATC map area classification.

11. For buildings which can resist the extreme ground motion expected at the site through large inelastic deformations, the use of infills like that of solid bricks reinforced externally with welded wire fabric offers considerable advantage over the use of just bare frame. Because these infilled frames can develop an interstory displacement ductility $\mu_\delta = 4.2$ with a reduction in strength of only 14%, the building can resist ground motions with an $a_{ep} \leq 0.44$ g even if only 4 of the 11 frames are infilled. To be able to resist a similar ground motion the bare frame building will need to develop a $\mu_\delta \geq 5.6$ with significantly larger displacement, and consequently more damage throughout the whole structure. In the case of infilled frame the damage will concentrate in just one or two stories.

Conclusions Drawn from Assessment of the Implication of Experimental Results Obtained Regarding the Repair and Retrofitting of Existing Buildings

1. For bare frames that have been damaged (cracking and spalling of unconfined concrete) due to considerable yielding, developing interstory displacement ductility of four, the following repair technique gives good result: removal of any crushed and loose concrete and recasting of it, and injection of cracks with epoxy.

2. Undamaged, or damaged bare frames after their repair, can be effectively retrofitted for seismic resistant purposes by the addition of reinforced infills that are properly attached (anchored) to the frame. Of all the infills studied, the one that offers the greatest potential to retrofit stiffness, strength and energy dissipation capacity to existing buildings is the one based on use of solid bricks reinforced externally with welded wire fabric covered with cement mortar and anchored to the frame, as illustrated in Fig. 4.

RECOMMENDATIONS FOR FUTURE RESEARCH

1. To investigate further the behavior of masonry infills which are externally reinforced with welded wire fabric and then covered with cement mortar or concrete. The use of soft hollow bricks or concrete blocks and of the shotcrete technique for applying the cover, should be studied.

2. New methods for attaching (anchoring) the infill panels to the frame in the case of retrofitting these panels to existing bare frames, should be investigated.

3. The values of the effective viscous damping ratio in bare frame and infilled frame building should be studied. The variation of this ratio as damage increases in the infills should also be investigated further.

4. Review the reliability of present analytical methods to predict strength, stiffness, and deformation capacity (energy absorption and energy dissipation capacities) of infilled frames and to develop new, simpler, and more reliable methods.

5. To conduct integrated analytical and experimental studies (using earthquake simulators) on the seismic response of buildings with infilled frames when they are subjected to different types of ground motions, particularly those including severe acceleration pulses of long duration.

6. To study effects of partial infilling as well as infill with openings.

7. To investigate the feasibility of using infills for taller buildings by studying ways of infilling (particularly the type of anchoraging) that will permit the spread of significant inelastic deformations to more than one story.

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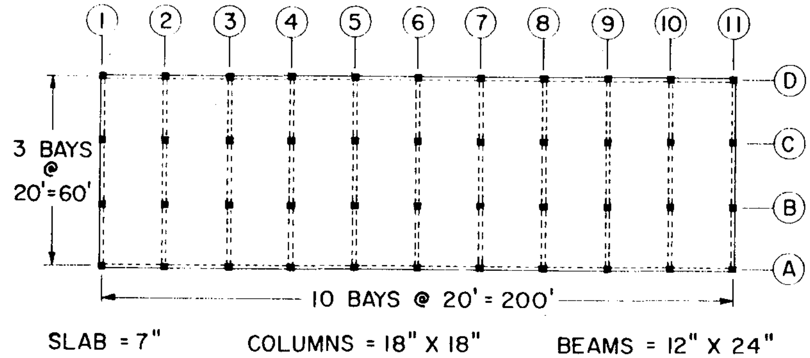
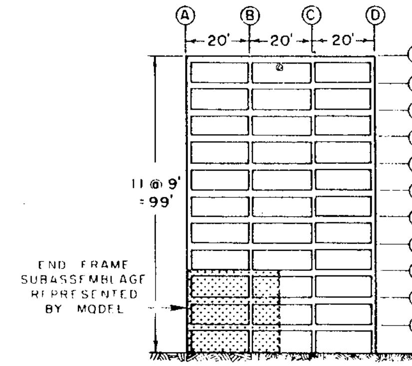
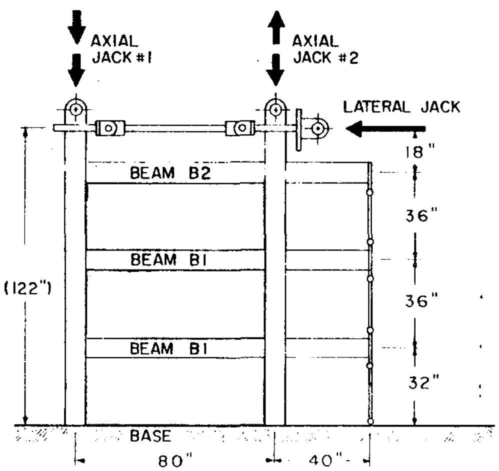


(a) FLOOR FRAMING PLAN

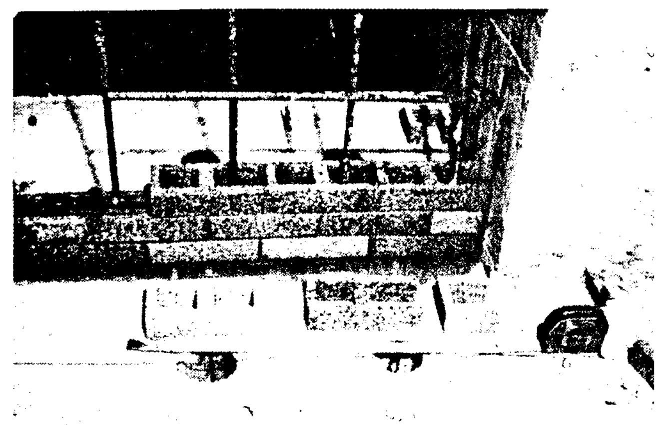


(b) ELEVATION OF PROTOTYPE FRAME

FIG. 1 PROTOTYPE BUILDING

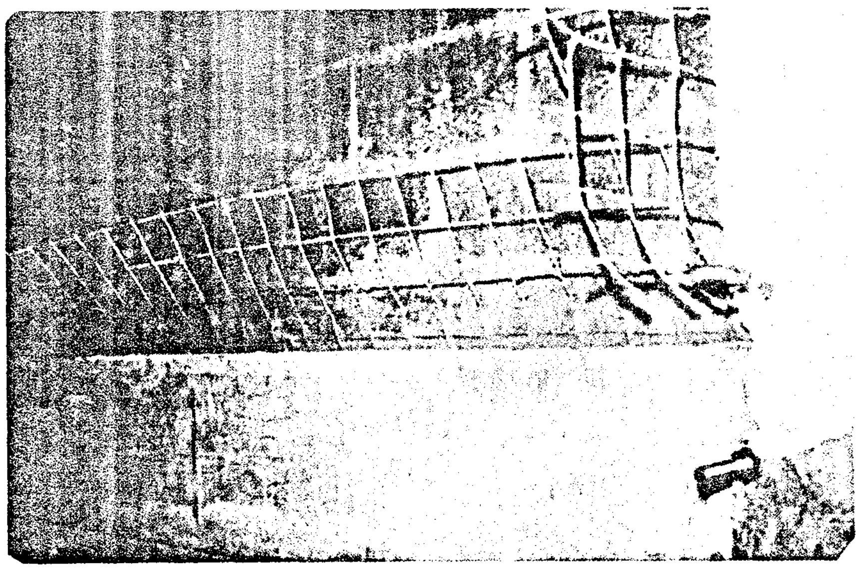


(a) 1/3-SCALE MODEL OF THE SUBASSEMBLAGE SHOWN IN FIG.1(b)

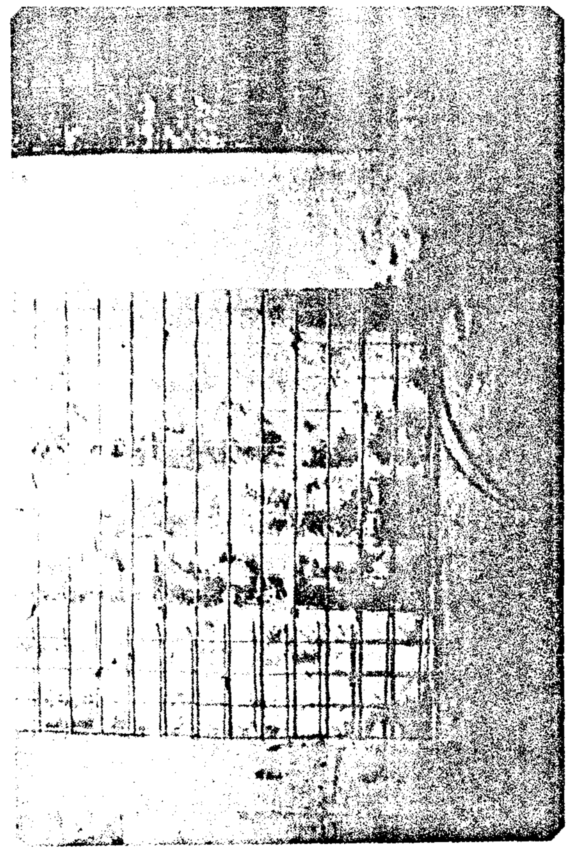


(b) REINFORCED HOLLOW BRICK INFILL

FIG. 2 TEST SPECIMENS

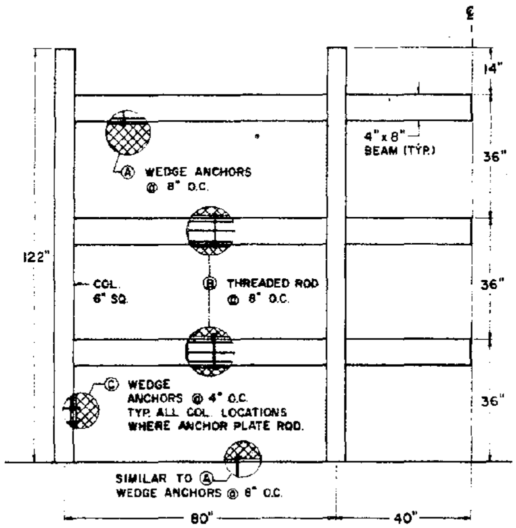


(a) WWF DOWELS IMBEDDED IN THE FRAME MEMBERS

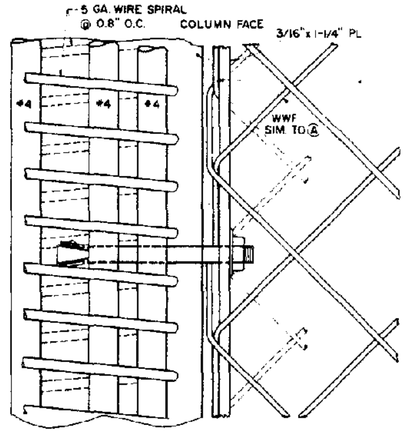


(b) WWF REINFORCEMENT AT EACH FACE OF BRICK INFILL

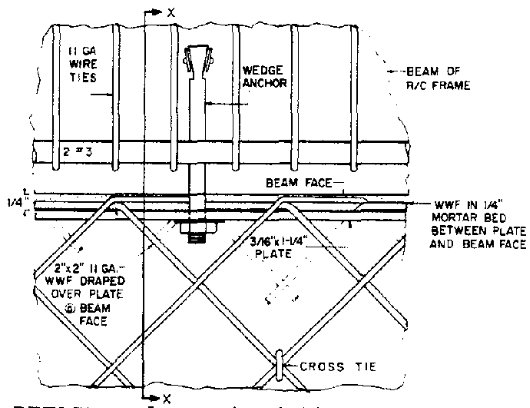
FIG. 3 DETAILS OF REINFORCED SOLID BRICK INFILLED FRAME: TEST SPECIMEN



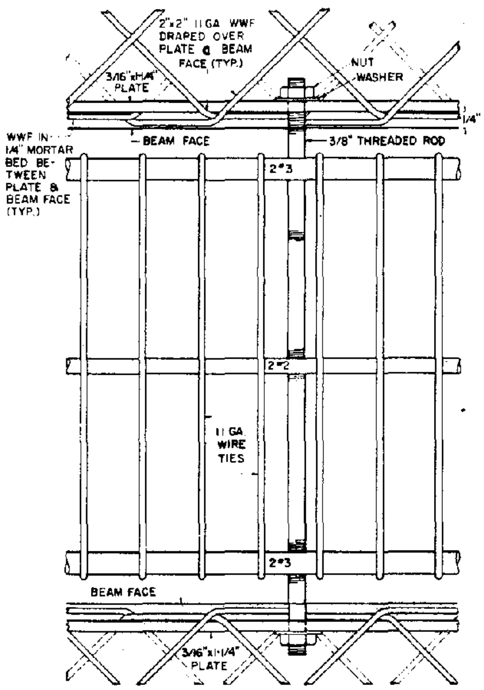
(a) FRAME-PANEL ANCHORAGE SYSTEM



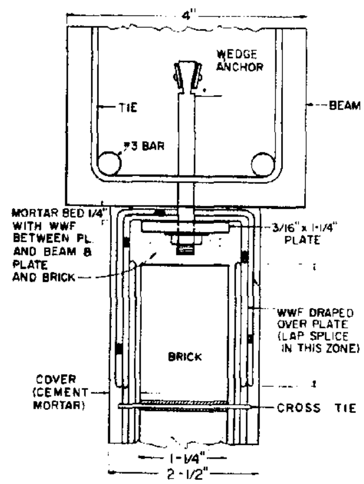
(c) DETAIL C [see Fig (a)], WEDGE ANCHOR FASTENING WWF TO COLUMN



(d) DETAIL A [see Fig (a)], WEDGE ANCHOR FASTENING WWF TO BEAM



(b) DETAIL B [see Fig (a)], THREADED ROD PROVIDING POSITIVE ANCHORAGE BOLTING COMPLETELY THROUGH BEAM



(e) SECTION X-X [see Fig (d)]

FIG. 4 DETAILS OF WWF REINFORCED INFILL USED TO RETROFIT EXISTING R/C BARE FRAME

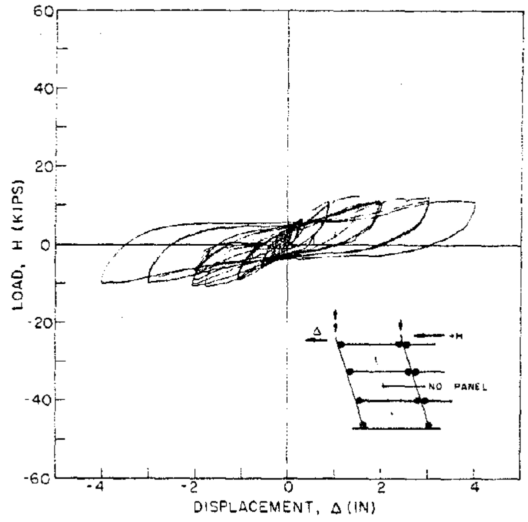


FIG. 5 LOAD DEFLECTION RELATIONSHIP FOR BARE FRAME: TEST SPECIMEN 15

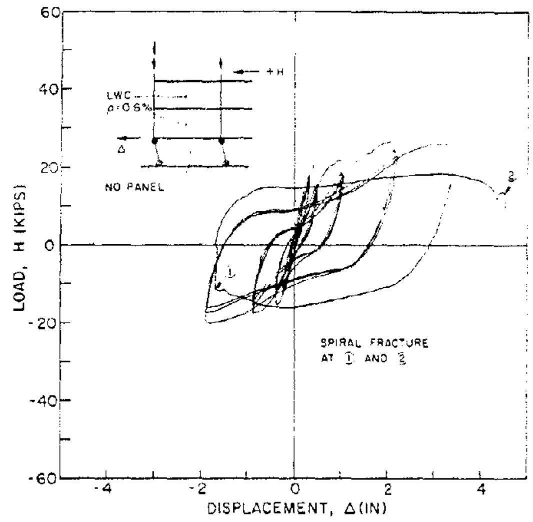
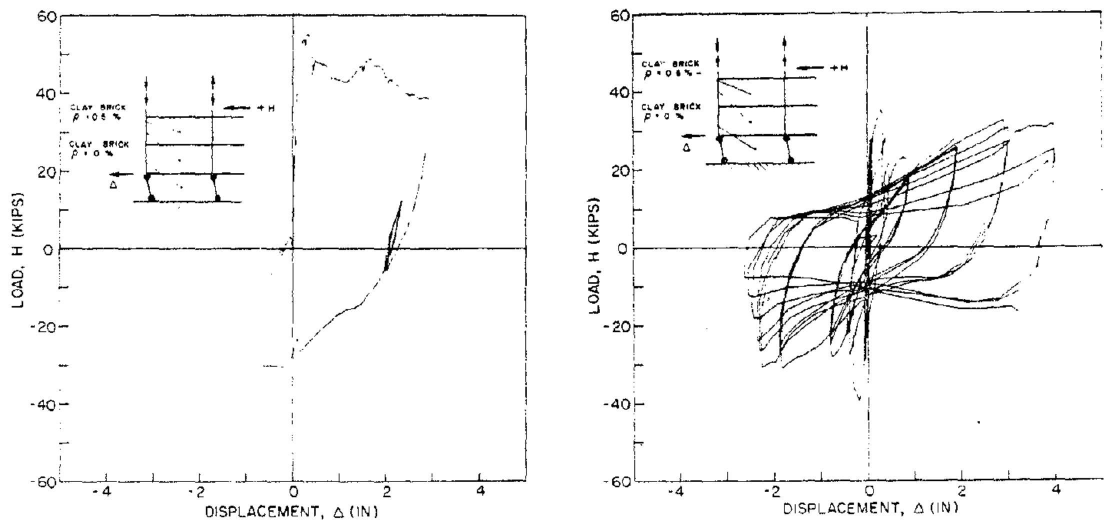
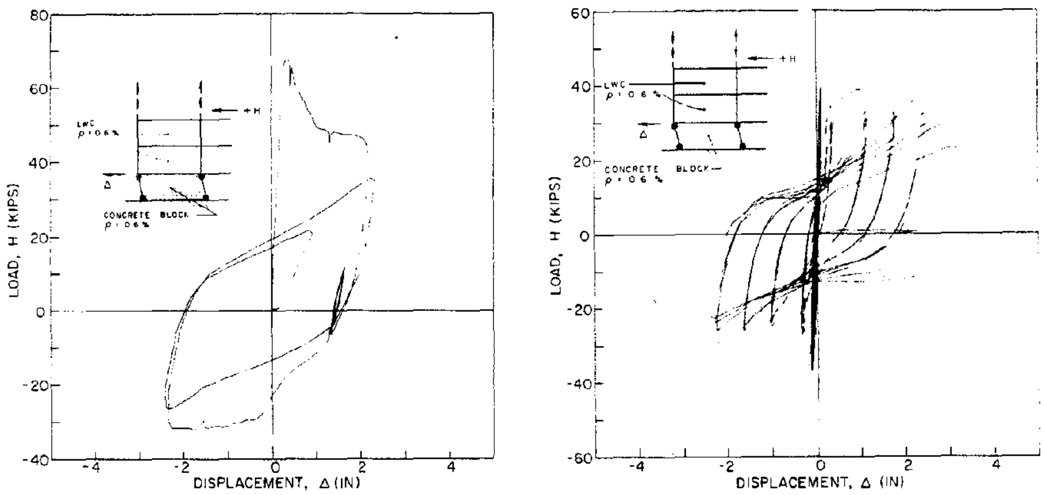


FIG. 6 LOAD DEFLECTION RELATIONSHIP FOR FIRST SOFT STORY FRAME: TEST SPECIMEN 16



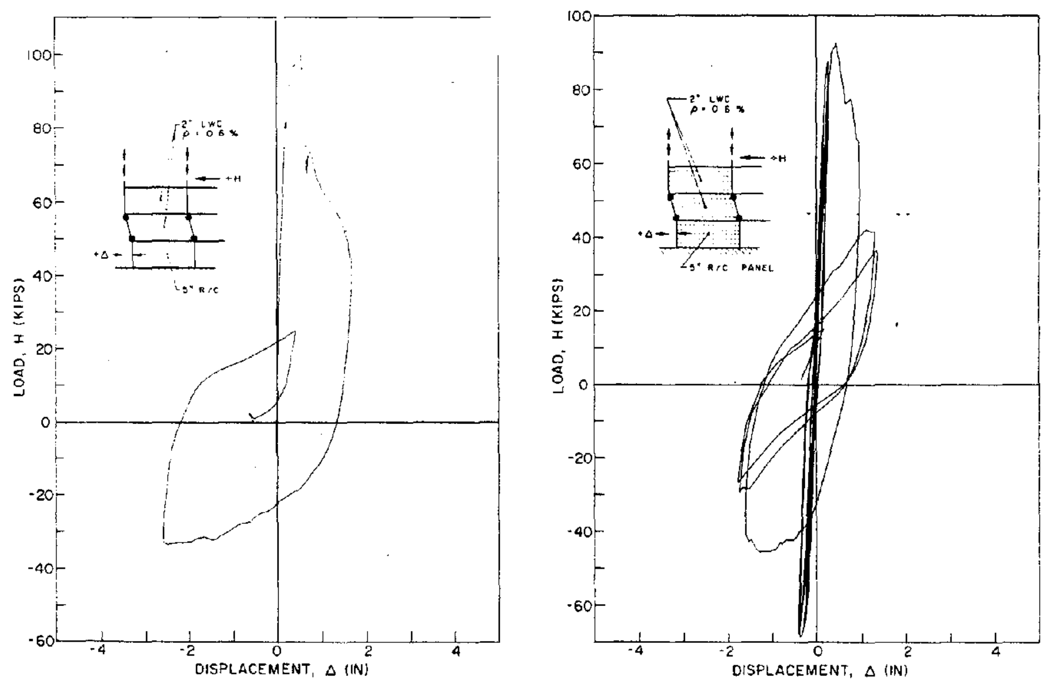
(a) (b)

FIG. 7 LOAD-DEFLECTION RELATIONSHIP FOR UNREINFORCED CLAY BRICK INFILL



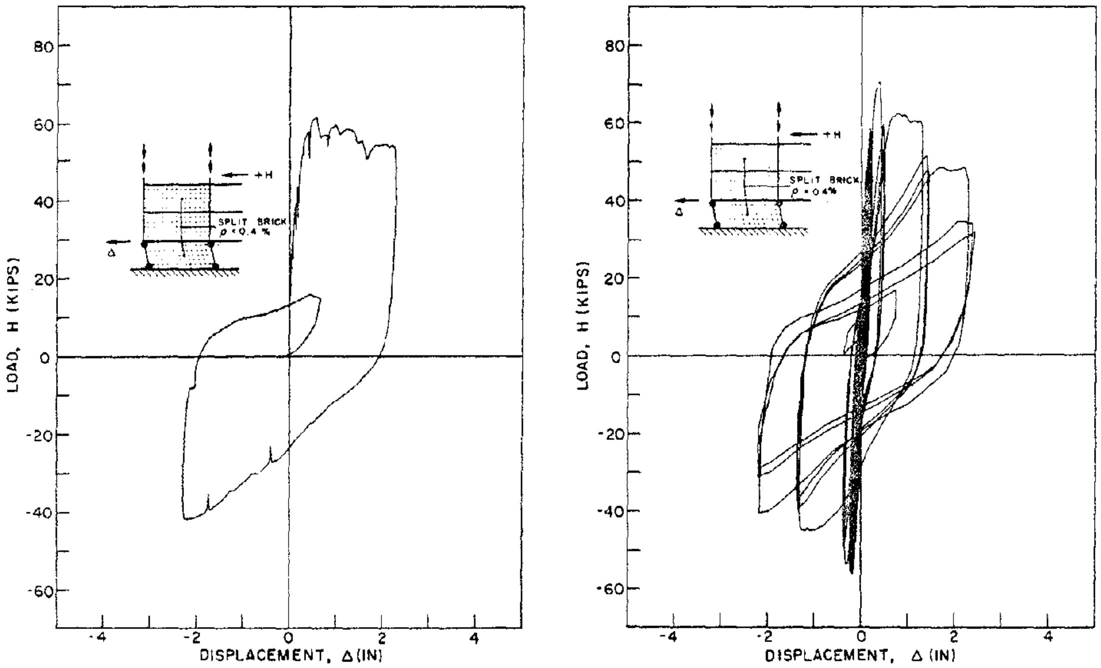
(a) MONOTONIC TEST: SPECIMEN 3 (b) CYCLIC TEST: SPECIMEN 8

FIG. 8 LOAD-DEFLECTION RELATIONSHIP FOR REINFORCED CONCRETE BLOCK INFILL



(a) MONOTONIC TEST: SPECIMEN 10 (b) CYCLIC TEST: SPECIMEN 11

FIG. 9 LOAD-DEFLECTION RELATIONSHIP FOR REINFORCED LIGHTWEIGHT CONCRETE



(a) MONOTONIC TEST: SPECIMEN 17 (b) CYCLIC TEST: SPECIMEN 16

FIG. 10 LOAD-DEFLECTION RELATIONSHIP FOR WWF REINFORCED BRICK INFILL

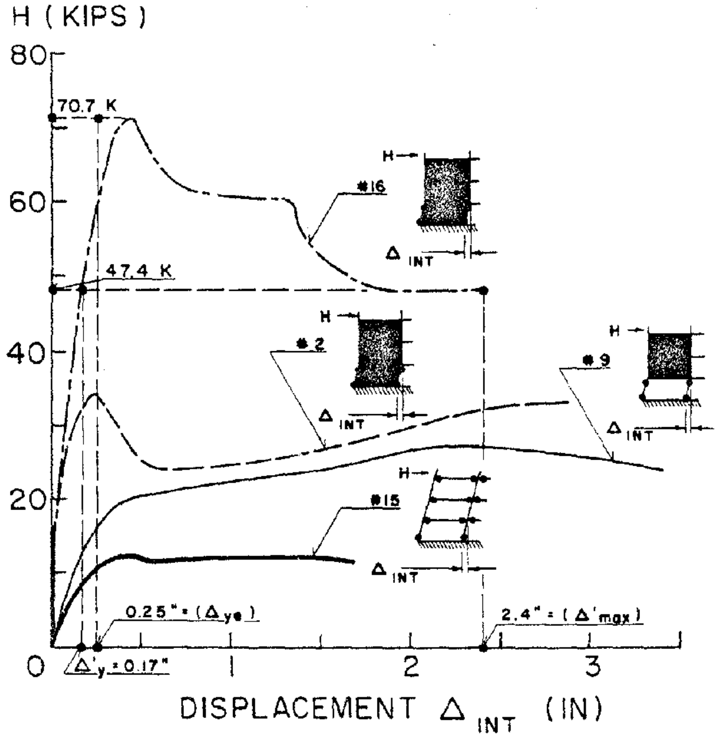


FIG. 11 LATERAL LOAD-INTERSTORY DRIFT DIAGRAMS FOR SOME SPECIMENS TESTED

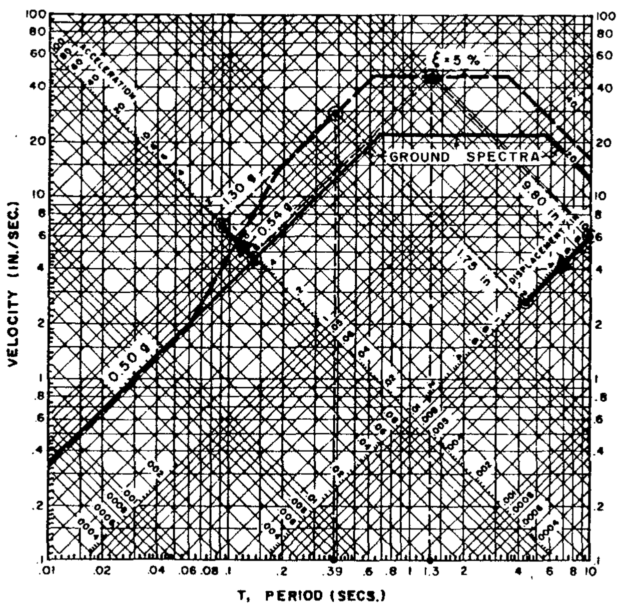


FIG. 12 SMOOTHED LINEAR ELASTIC RESPONSE SPECTRA FOR AN EFFECTIVE PEAK GROUND ACCELERATION OF 0.5g AND A DAMPING RATIO ξ = 5% AFTER NEWMARK AND HALL [8]. ILLUSTRATION OF EFFECTS OF CHANGES IN T ON FORCE AND DEFORMATION DEMANDS.

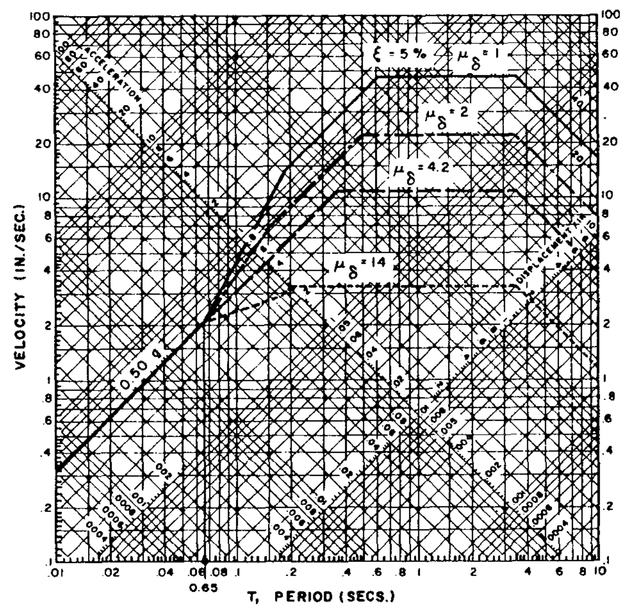


FIG. 13 SMOOTHED INELASTIC DESIGN RESPONSE SPECTRA FOR DIFFERENT VALUES OF THE DISPLACEMENT DUCTILITY, μ_δ, DERIVED FROM THE LINEAR DESIGN SPECTRA OF FIG. 12.

TABLE 1 SUMMARY OF SPECIMENS TESTED AND THEIR MAXIMUM RESISTANCE

TEST SPECIMEN NO.	MODEL NO.	LOADING PROGRAM	FIRST-STORY PANEL	SECOND-STORY PANEL	THIRD-STORY PANEL	MAX LOAD H (KIPS)*	LOCATION OF FAILURE
1	1	Monotonic	Clay Brick ρ=0%	Clay Brick ρ=0.6%	Clay Brick ρ=0.6%	55.2	First Story
2	1,R1	Cyclic	Clay Brick ρ=0%	Clay Brick ρ=0.6%	Clay Brick ρ=0.6%	35.3	First Story
3	3	Monotonic	Concrete Brick ρ=0.6%	LWC ρ=0.6%	LWC ρ=0.6%	67.9	First Story
4	2	Cyclic	Clay Brick ρ=0.6%	Clay Brick ρ=0.6%	Clay Brick ρ=0.6%	54.5	First Story
5	1,R2	Monotonic	6in. R/C	Clay Brick ρ=0.6%	Clay Brick ρ=0.6%	68.6	Third Story
6	1,R3	Cyclic	6in. R/C	Clay Brick ρ=0.6%	RC ρ=0.6%	80.0	Second Story
7	2,R1	Cyclic	Clay Brick ρ=0.15%	Clay Brick ρ=0.6%	Clay Brick ρ=0.6%	39.2	First Story
8	3,R1	Cyclic	Concrete Brick ρ=0.6%	LWC ρ=0.6%	LWC ρ=0.6%	46.7	First Story
9	3,R2	Cyclic	No Panel	LWC ρ=0.6%	LWC ρ=0.6%	27.4	First Story
10	3,R3	Cyclic	6in. R/C	LWC ρ=0.6%	LWC ρ=0.6%	92.7	Second Story
11	3,R4	Monotonic	6in. R/C	LWC ρ=0.6%	LWC ρ=0.6%	100.0	Second Story
12	1,R4	Monotonic	6in. R/C	Clay Brick ρ=.15%	RC ρ=0.6%	63.2	Second Story
13	2,R2	Cyclic	6in. R/C	Clay Brick ρ=0.6%	Clay Brick ρ=0.6%	76.0	Third Story
14	2,R3	Monotonic	6in. R/C	Clay Brick ρ=0.6%	RC ρ=0.6%	83.0	Second Story
15	4	Cyclic	No Panel	No Panel	No Panel	12.6	Total Mechanism
16	5	Cyclic	Split Brick 90° =0.4%	Split Brick 90° =0.4%	Split Brick 90° =0.4%	70.7 56.6**	First Story
17	5,R1	Monotonic	Split Brick 90° =0.4%	Split Brick 90° =0.4%	Split Brick 90° =0.4%	61.3 49.0**	First Story
18	4,R1	Cyclic	Split Brick 45° =0.4%	Split Brick 45° =0.4%	Split Brick 45° =0.4%	57.3 45.8**	Combined Mechanism

*1 Kip = 4.45 KN

** Factored by $\frac{2.0\ \text{in}}{2.5\ \text{in}}$

TABLE 2 MAXIMUM INTERSTORY LATERAL STIFFNESS OF TEST SPECIMENS

TEST SPECIMEN	MAXIMUM INTERSTORY LATERAL STIFFNESS		
	INITIAL TANGENT (K/in)*	EFFECTIVE STIFFNESS AT SERVICE LOAD LEVEL, K_I^S (K/in)*	RELATIVE STIFFNESS K_I^S/K_{bf}^S
1	1090	206	5.89
2	1090	236	6.74
3	585	212	6.06
4	920	187	5.34
5	195	195	5.57
6	271	238	6.80
7	780	195	5.57
8	725	250	7.14
9	103	60	1.71
10	990	358	10.23
11	1500	409	11.69
12	494	167	4.77
13	178	176	5.03
14	203	210	6.00
15	65	35	1.00
16	1250	292(234)	8.34(6.69)**
17	834	118(94)	3.37(2.69)**
18	960	203(162)	5.80(4.63)**

*1 K/in = 0.175 KN/mm ** Factored by $\frac{2.0 \text{ in}}{2.5 \text{ in}}$

TABLE 3 EFFECTS OF INFILLS ON THE PERIOD, T_{if}, OF THE PROTOTYPE BUILDING

DEGREE OF CHANGES AND TYPE OF INFILL	UPPER BOUND ALL 11 FRAMES ARE INFILLED				LOWER BOUND ONLY 4 OF THE 11 FRAMES ARE INFILLED			
	SAME MASS			INFILL ADDS MASS	SAME MASS			INFILL ADDS MASS
	T^+_{if}/T_{bf}*	T^+_{if} in secs for T_{bf} (secs) 1.30	T^+_{if} in secs for T_{bf} (secs) 1.01	T^+_{if}/T_{bf}	T^-_{if}/T_{bf}	T^-_{if} in secs for T_{bf} (secs) 1.30	T^-_{if} in secs for T_{bf} (secs) 1.01	T^-_{if}/T_{bf}
LOWEST (Solid Brick with WWF)	0.46	0.60	0.46	0.49	0.65	0.84	0.66	0.66
AVERAGE (Hollow Massonry)	0.40	0.52	0.40	0.42	0.58	0.75	0.59	0.59
HIGHEST (Lightweight Concrete)	0.30	0.39	0.30	0.32	0.47	0.61	0.47	0.48

*T_{bf} is the Period of the Prototype Building with Bare Frame Structure

TABLE 4 EFFECTS OF INFILLS ON SUPPLIED MAXIMUM STRENGTH OF THE PROTOTYPE BUILDING, $(V_n)_{if}$

COMPARISON OF $(V_n)_{if}$ WITH THE SUPPLIED MAXIMUM STRENGTH OF THE BUILDING BARE FRAME $(V_n)_{bf}$ BASED ON THE MEASURED STRENGTH OF THE SPECIMENS, i.e., $\frac{(V_n)^s_{if}}{(V_n)^s_{bf}}$ BEING $(V_n)^s_{bf} = 12.5$ kips

TYPE OF INFILL AND REINFORCEMENT	(ρ in%)	UPPER BOUND ALL 11 FRAMES ARE INFILLED: $(V_n)^s_{if}$ (kips)*	$(V_n)^s_{if}/(V_n)^s_{bf}$	$(V_n)_{if}$ kips	Increase of Strength in %	LOWER BOUND ONLY 4 OF 11 FRAMES ARE INFILLED: $\frac{7(V_n)^s_{bf}+4(V_n)^s_{if}}{11(V_n)^s_{bf}}$	$(V_n)_{if}$ kips	Increase of Strength in %
UNREINFORCED MASONRY	0.	Lower 35.3	2.82	6989	182	Highest 1.66	4117	66
						Lowest 1.34	3329	34
REINFORCED HOLLOW MASONRY	0.15	Lower 39.0	3.14	7722	214	Highest 1.78	4383	78
	0.60					Lowest 1.46	3596	46
		Lowest 46.7	3.74	9247	274	Highest 2.00	4937	100
						Lowest 1.68	4150	68
		Average 65.0	5.20	12870	420	Highest 2.53	6255	153
						Lowest 2.21	5468	121
		Highest 83.0	6.64	16434	564	Highest 3.05	7551	205
						Lowest 2.73	6764	173
SOLID BRICK REINFORCED WITH WWF	0.40	Lowest 57.3	4.58	11345	358	Highest 2.30	5701	130
						Lowest 1.39	4914	98
		Average 63.1	5.05	12494	405	Highest 2.47	6118	147
						Lowest 2.15	5330	115
		Highest 70.7	5.65	13999	465	Highest 2.69	6665	169
						Lowest 2.37	5878	137
REINFORCED LIGHTWEIGHT CONCRETE	0.60	Lower 92.7	7.42	18355	642	Higest 3.34	8249	234
						Lowest 3.02	7452	202
		Higher 100.0	8.00	19800	700	Highest 3.55	8775	255
						Lowest 3.23	7988	223

*1 K = 4.45 KN

TABLE 5 INCREASE IN LINEAR ELASTIC SEISMIC FORCE DEMANDS, V^D_{if}, DUE TO CONSIDERATION OF INFILLS AS STRUCTURAL ELEMENTS

COMPARISON OF V^D_{if} WITH SEISMIC FORCE DEMANDS BASED ON THE BUILDING BARE FRAME STRUCTURE, V^D_{bf}, AND FOR SAME MASS

DEGREES OF CHANGE IN T, AND TYPE OF INFILL	T_{bf} in secs	UPPER BOUND ALL 11 FRAMES ARE INFILLED: T^+_{if} in secs	V^D_{in}/V^D_{bf}	Increase in %	LOWER BOUND ONLY 4 OF THE 11 FRAMES ARE INFILLED: T^-_{if} in secs	V^D_{in}/V^D_{bf}	Increase in %
LOWEST (Solid Brick with WWF)	$\frac{1.30}{1.01}$	$\frac{0.60}{0.46}$	$\frac{2.41}{1.86}$	$\frac{141}{86}$	$\frac{0.84}{0.66}$	$\frac{1.56}{1.57}$	$\frac{56}{57}$
AVERAGE (Hollow Masonry)	$\frac{1.30}{1.01}$	$\frac{0.52}{0.40}$	$\frac{2.41}{1.86}$	$\frac{141}{86}$	$\frac{0.75}{0.54}$	$\frac{1.76}{1.86}$	$\frac{76}{86}$
HIGHEST (Lightweight Concrete)	$\frac{1.30}{1.01}$	$\frac{0.39}{0.30}$	$\frac{2.41}{1.86}$	$\frac{141}{86}$	$\frac{0.61}{0.47}$	$\frac{2.41}{1.86}$	$\frac{141}{86}$

TABLE 6 DECREASE IN LINEAR ELASTIC DISPLACEMENT DEMANDS, δ_{if}^{D}, DUE TO CONSIDERATION OF INFILLS AS STRUCTURAL ELEMENTS

COMPARISON OF δ_{if}^{D} WITH THE DISPLACEMENT DEMANDS BASED ON THE BUILDING BARE FRAME STRUCTURE, δ_{bf}^{D}

DEGREES OF CHANGE IN T, AND TYPE OF INFILLS	T_{bf}^{P} in secs	UPPER BOUND ALL 11 FRAMES ARE INFILLED			LOWER BOUND ONLY 4 OF THE 11 FRAMES ARE INFILLED		
		T_{if} in secs	$\delta_{in}^{D}/\delta_{bf}^{D}$	Decrease in %	T_{if} in secs	$\delta_{in}^{D}/\delta_{bf}^{D}$	Decrease in %
LOWEST (Solid Brick with WWF)	1.30 / 1.01	0.60 / 0.46	0.44 / 0.34	56 / 66	0.84 / 0.60	0.66 / 0.67	34 / 33
AVERAGE (Hollow Masonry)	1.30 / 1.01	0.52 / 0.40	0.34 / 0.24	66 / 76	0.75 / 0.54	0.60 / 0.49	40 / 51
HIGHEST (Lightweight Concrete)	1.30 / 1.01	0.39 / 0.30	0.18 / 0.15	82 / 85	0.61 / 0.47	0.46 / 0.40	54 / 60

TABLE 7 BUILDING SEISMIC RESISTANT COEFFICIENT, $C = \frac{(V_n)}{W}$ AND EFFECTIVE PEAK ACCELERATION, a_{ep}, THAT IT CAN RESIST ELASTICALLY

TYPE OF INFILL AND REINFORCEMENT	(ρ in %)	UPPER BOUND ALL 11 FRAMES ARE INFILLED				LOWER BOUND ONLY 4 OF 11 FRAMES ARE INFILLED			
		V_n (kips)*	C	T (secs)	a_{ep}/g	V_n (kips)*	C	T (secs)	a_{ep}/g
NONE BARE FRAME		2475	0.11	1.30	0.10	2475	0.11	1.30	0.10
UNREINFORCED MASONRY	0.%	6989 (L)	0.30	0.52	0.12	3329	0.14	0.75	0.07
REINFORCED HOLLOW MASONRY	0.15%	7762 (L)	0.34	0.52	0.13	3610	0.16	0.75	0.08
	0.6%	12870 (Av)	0.56	0.52	0.22	5468	0.24	0.75	0.13
SOLID BRICK REINFORCED WITH WWF	0.4%	12494 (Av)	0.54	0.60	0.21	5330	0.23	0.84	0.14
REINFORCED LIGHTWEIGHT CONCRETE	0.6%	19107 (Av)	0.83	0.39	0.32	7735	0.33	0.61	0.17

*1 kip = 4.45 KN

APPENDIX 1. REFERENCES

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10. Priestley, M. J. N., "Masonry," Design of Earthquake Resistant Structures, E. Rosenblueth, Ed., John Wiley & Sons, New York, N.Y., 1980, pp. 195-222.

APPENDIX II. NOTATION

The following symbols are used in this paper:

a = acceleration

K = lateral stiffness

L = length

M = mass

P = axial load

r = number of the story to the top

T = period

V_n = base shear strength

ρ = percentage of main reinforcing steel

ϕ = curvature

ξ = effective viscous damping ratio

μ = ductility ratio

Δ = lateral displacement

Subscripts

bf = bare frame

ep = effective peak

I = interstory

if = infilled frame

rif = reinforced infilled frame

s = scale

uci = ultimate curvature at section i

yci = yielding curvature at section i

δ = displacement

Superscripts

D = demands

m = model frame

p = prototype frame

s = specimen