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R.D. Hanson

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Papers delivered at the seminar are presented. They address the following topics: seismic strengthening of old buildings with modern codes; aseismic strengthening of reinforced concrete buildings; repair and retrofit of existing steel building structures; retrofitting bridges to increase their seismic resistance; fire testing of epoxy repaired shear walls; and ways to evaluate the seismic safety of houses. It is concluded that repair and rehabilitation techniques for seismically weak and historical buildings are different; the consequences of these differences may have direct application to the design of new buildings to accommodate future strengthening. It is acknowledged that field inspection of construction and workmanship of construction craftsmen have a major influence on the seismic capabilities of completed buildings. It is recommended that a means to improve the quality of workmanship and inspection be developed.

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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

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

Proceedings of the First Seminar—May 1980

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INTRODUCTION

The first joint meeting of the US/JAPAN Cooperative Research Program in Earthquake Engineering on Repair and Retrofit of Buildings and Lifelines was held in Los Angeles from May 15 through May 17, 1980. Although the original title includes Lifelines it was decided by the participants to limit the presentations and discussions at this meeting and future meetings to the structural aspects of lifelines. Therefore it was considered appropriate to change the name of this series of meetings from "Buildings and Lifelines" to "Structures."

The meeting schedule was established to provide technical presentations and discussions on Friday, May 16 and field site visits to various examples of repair and retrofitting on Saturday, May 17. It was felt that this schedule would provide the maximum amount of interaction within the two day meeting. The technical presentations and discussions were held at Agbabian Associates offices, 250 North Nash Street, El Segundo, California and the site visits extended from Long Beach to Pasadena. The following program provides more detail as to these activities and lists the individual making the presentations. Some of the scheduled participants were not able to attend the meeting so their papers were presented by the identified person.

Official representatives attending from Japan included Dr. Toshio Iwasaki, Public Works Research Institute; Dr. Shunsuke Sugano, Takenaka Komuten Co., and Dr. Makoto Watabe, Building Research Institute. Official representatives attending from the United States included M. S. Agbabian, Agbabian Associates; Professor Vitelmo Bertero, University of California at Berkeley; Oris Degenkolb, CALTRANS; G. R. Fuller, Housing and Urban Development; Robert D. Hanson, University of Michigan; H. S. Lew, National Bureau of Standards; John Meehan, California State Architect's Office; Joseph Plecnik, Long Beach State University; John Scalzi, National Science Foundation; James Warner, consultant; and Loring Wyllie, Jr., H. J. Degenkolb & Associates. A limited number of observers from Japan and the United States attended the presentations and discussions or portions of the field trip.

PROGRAM

Thursday, May 15, 1980
- 5:30 p.m. - Group Gathering and Dinner at Haji Baba's

Friday, May 16, 1980 - Technical Presentations and Discussions
- 8:30-9:00 a.m. - Opening Session Cochairmen: Watabe and Hanson
- 9:00-12:00 m. - Session I Co-chairmen: Iwasaki and Agbabian
  - Seismic Strengthening of Old Buildings with Modern Codes by Wyllie

Aseismic Strengthening of Existing Reinforced Concrete Buildings by Sugano

Overview of U.S. Experiences - Current Practice and Weaknesses by Warner

Retrofitting and Repairs of Existing Steel Structures by Watabe

Seismic Evaluation and Rehabilitation of HUD Residential Buildings by Fuller

Repair and Retrofit Project by Wooden Houses by Watabe

12:00-2:00 p.m. - Lunch

Antioch High School Roof Collapse by Meehan

2:00-5:00 p.m. - Session II Co-chairmen: Sugano and Lew
Inspection and Retrofitting for Earthquake Resistance
Vulnerability of Highway Bridges in Japan by Iwasaki

Retrofitting Bridges to Increase their Seismic Resistance by Degenkolb

Repair and Retrofit Works for Existing Highway Bridges by Iwasaki

Preliminary Report on Fire Testing of Epoxed Repaired Shear Walls by Plecnik

Research Project on Repair and Retrofit of Buildings by Watabe

Repair of Bond in R/C Structures by Epoxy Injection by Bertero

5:00-5:30 p.m. - Closing Session

7:15 p.m. - Group Dinner at Marina City Club

Saturday, May 17, 1980
Session III - Repair and Retrofit Field Trip to Bridge and Building Sites
8:30 a.m. - Leave Hacienda Hotel
Visit CALTRANS Bridge sites in the Terminal Island area

11:30 a.m. - Arrive at Marshall High School, Los Angeles
Lunch and inspection of building

2:00 p.m. - Arrive at Jet Propulsion Laboratory, Pasadena
Discussion and inspection of rehabilitation of steel and reinforced concrete buildings

6:00 p.m. - Reception at J. Warner's home in La Canada
Meet with local engineers

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SUMMARY AND RESOLUTIONS

The first joint meeting was held in Los Angeles on May 15-17, 1980 with active participation by the individuals listed in the Introduction. Technical presentations were made on May 16 and field visits to bridge and building repair and rehabilitation sites were made on May 17 according to the schedule summarized in the Program. The following observations and recommendations were made:

1. Through the presentations and discussions of prepared papers, a mutual understanding of similar problems and solutions for the repair and retrofit of structures has been achieved. It was concluded that this first meeting was successful.

2. It was concluded that this Joint Committee should continue to concentrate its activities in the area of structures.

3. It was resolved that the following tasks should be accomplished prior to the next joint committee meeting:

   (a) exchange of materials on the establish procedures and practices for the evaluation and rehabilitation of buildings and bridges for natural hazard mitigation. These materials should be translated into the respective languages for a deeper understanding by the relevant fields;

   (b) summarize and compare current US and Japanese procedures and practices;

   (c) solicit from practicing professionals and others active in this field a summary of problems encountered in developing repair and retrofit designs and construction;

   (d) collect and exchange data on field experiences with jacking repair of buildings and bridges;

   (e) focus attention on the development of practical repair and retrofit techniques for low rise building structures.

4. It was recognized that repair and rehabilitation techniques for seismically weak and historical buildings are different. The consequence of these differences could have direct application to the design of new buildings to accommodate future strengthening.

5. It was acknowledged that field inspection of construction and workmanship of construction craftsmen have a major influence on the seismic capabilities of completed buildings. It is recommended that a means to improve the quality of workmanship and inspection be developed. This may be achieved through educational programs or literature.

6. Specific needed research activities were discussed. Due to the lack of sufficient time it was not possible to itemize or prioritize these activities.

7. It was acknowledged that this exchange of ideas, problems and solutions was to the mutual benefit of both Japan and the USA. It is recommended that this activity be continued for several years and that the next joint committee meeting be held in Japan in May, 1981.

SEISMIC STRENGTHENING OF OLD BUILDINGS WITH MODERN CODES

Loring A. Wyllie, Jr.
Structural Engineer
H.J. Degenkolb & Associates
San Francisco, California USA

for presentation at
US/Japan Cooperative Research Program in
Earthquake Engineering on Repair and
Retrofit of Buildings and Lifelines

May 16, 1980

INTRODUCTION

Structural strengthening of old buildings for seismic forces is frequently required by governmental authorities or desired by building owners. The reason for the strengthening may be to strengthen the building after an earthquake, to mitigate a hazardous condition, to permit changed or increased occupancy of the building or as part of an overall rehabilitation of the building.

Building regulations or Building Codes adopted by local governmental agencies in the United States are generally based on a model code such as the Uniform Building Code (Reference 1). These codes are written for new construction using modern materials and construction techniques. They do not include provisions for the old or archaic materials of construction which are present in old buildings which are to be remodeled or strengthened. Thus, the Building Code does not provide guidance to the Engineer nor criteria for the Building Official in evaluating the available strength of the building for lateral forces.

This paper attempts to summarize several approaches to strengthen buildings that have recently been utilized in the seismically active areas of the United States. These approaches are not a complete answer to the situation but they do highlight several of the issues that are present. It remains that considerable sound engineering judgment is required in the lateral strengthening of old buildings in seismic areas.

AN APPROACH FOR BUILDINGS WITH INHERENTLY SOUND SEISMIC RESISTANCE - Structural strengthening of old buildings for seismic forces must be approached with an understanding of how buildings of that era were designed. For example, significant buildings of six to twenty stories built in San Francisco in the late 1800's and early 1900's generally had a structural steel frame designed for wind forces and substantial cladding or walls of brick or nominally reinforced concrete. These buildings survived the 1906 San Francisco earthquake with virtually no structural damage, some cracking and damage to the brick or concrete walls, and considerable fire damage from the fire which followed the earthquake. A committee of Structural Engineers of the San Francisco Section American Society of Civil Engineers, after studying the building performance in 1906, concluded that a building properly designed for a 30 pound per square foot (1.44 $_k P_a$) wind load should survive an earthquake like the 1906 event (Reference 2). When we consider the high seismic design forces of today, the 30 psf value seems ridiculously low. However, the key phrase is a "properly designed" building. Properly designed in 1906 really meant that the steel frame was designed for wind loads and the brick and concrete wall, anchored to the steel frame, was not calculated but added considerable stiffness and strength to the lateral force resisting system.

Furthermore, the weight of this brick and concrete was generally supported by the steel frame so building stability was in no way threatened if the brick or concrete cracked. These buildings performed well in 1906, and provided their materials have not deteriorated from weathering or corrosion, and provided they have not been extensively remodeled, they should still be accepted as seismic resistant structures without the need to comply to modern building codes. However, that is generally not recognized by the working of our modern codes.

An example of a recent rehabilitation project of a building of this type is the former Hotel Oakland. This eight-story building has the shape of a U and occupies a city block in downtown Oakland, Figure 1 is an exterior view of the building near its main entrance. The building was originally a hotel and had been converted to a government hospital. It has been remodeled for housing for the elderly funded by the U.S. Government Department of Housing and Urban Development (HUD). HUD requires such projects to conform to minimum seismic requirements but requires conformance to less than modern code requirements when engineering judgment confirms good seismic performance with resistance provided to prevent major collapse of loss of life due to earthquake forces. The Hotel Oakland has reinforced concrete floor slabs and structural steel framing. The structural steel frame has riveted clip angle connections similar to those of the St. Francis Hotel in San Francisco that survived the 1906 San Francisco earthquake.

Rehabilitation of the Hotel Oakland involved removal of all the interior unreinforced hollow clay tile partitions which have proved so hazardous in past earthquakes and replacing them with modern partition systems. The exterior brick walls were investigated and found to be unreinforced but sound with good quality mortar and of substantial proportions. As their shear capacity was to be relied upon for lateral forces, anchors and vertical studs were added to prevent their failure perpendicular to the plane of the wall. Figure 2 illustrates the condition on a typical floor where heavy duty metal studs used for furring had a connecting steel member installed which fastened to a bolt embedded in the brick work with epoxy. This detail was repeated at frequent intervals. Figure 3 illustrates a similar condition in the Ground Floor where a high story height required a wide flanged structural steel member spanning between floors with the epoxy bolts into the brickwork.

New reinforced concrete walls designed to resist reasonable lateral forces were added at elevator and stair shafts in locations to minimize torsion. In the lower stories where brick walls above terminated over the enlarged first story lobbies and ballrooms, new concrete shear walls were also added to remove the discontinuity of stiffness that existed in the Ground Floor. The design criteria was for about 4% g which represented 60% of the requirements of the 1973 Edition of the Uniform Building Code. At this force level, the braced but unreinforced brick walls resist about half the shear at a stress of 12 to 18 psi (82.7 to 124 kPa). The added concrete walls were designed to current code criteria for the remainder of the shear and had hammerheads in the basement for increased stability for overturning forces. Our most recent code (the 1979 Edition of the Uniform Building Code) would indicate a requirement for 6 to 9% g for lateral force design, depending on the soil factor determined. This rehabilitation approach was accepted by HUD as well as the City of Oakland Building Department and should provide reasonable safety to the occupants of the building. It must be emphasized that only with the cooperation of the building authorities was this judgmental approach feasible.

AN APPROACH FOR SEISMICALLY WEAK BUILDINGS -

Many old buildings do not have the inherent strength and good performance record of the previous category. Examples prevalent in California of this category of buildings are typified by the structure with exterior unreinforced brick bearing walls with weak lime mortar and wood floors and roof with straight board sheathing. Anchorage between the wood and brick is generally poor and collapse has a high probability in strong ground shaking. These buildings are frequently rehabilitated as they possess an historic atmosphere for restaurants or small offices and rehabilitation is usually feasible economically. As the brick mortar is weak, it is usually most prudent to provide a new and independent bracing scheme for the building complying with current codes. This is usually done by providing new braced steel frames for horizontal shears with new plywood added to create reliable horizontal diaphragms. Positive anchorage of the brick walls to the strengthened structural system is also necessary.

An example of this type of strengthening is an old two-story firehouse which was recently investigated. The building was narrow and long with high solid property line brick walls on the long sides and virtually all doors and windows at each short end. The proposed strengthening scheme added three or four rigid frames of structural steel in the transverse direction and spaced to minimize the stresses in the new plywood floor and roof diaphragms. Vertical structural steel members were proposed at 6 to 10 foot centers against the brick walls between floors with anchors in the brick to stabilize it for forces perpendicular to the wall. These vertical members connected to new continuous horizontal steel members which serve as diaphragm chords as well as providing the connection to the diaphragm. Several diagonal braces were added along the side walls to provide new bracing in the longitudinal direction. Special bracing was required for the decorative brick and stone work on the front of the building. Thus, a new independent bracing system was provided for the building with the weak unreinforced brick walls providing redundant strength to their ability.

A CODE FOR HISTORICAL STRUCTURES -

Buildings, structures and places of historic interest in the United States are designated by placing them in a Register of Historic Places, or inventory of historic places, either by the federal, state or local level of government. This procedure is an attempt to preserve buildings of historic significance, either because of the historic usage of the building or because of the architectural treatment of the building. When restoring buildings of historic significance, there are many factors, such as seismic resistance, fire safety, exiting requirements, access for the handicapped, etc., where compliance with modern codes might destroy the historical fabric of the building which is trying to be preserved. In an attempt to remedy this situation, the State of California recently adopted a State Historical Building Code (Reference 3). Although experience has to be gained to review the effectiveness of this new Code, it does appear to be a good beginning in a difficult area.

The structural requirements of California's Historical Building Code require a survey and evaluation of the building by a Structural Engineer or an Architect knowledgeable in earthquake resistant design. The building is analyzed using seismic forces as specified in the current code but allowing consideration of ultimate capacities of the materials and broad judgment regarding the strength of materials not recognized by prevailing codes. The Historical Code also contains a chapter on Archaic Materials and Methods of Construction which gives guidance on establishing allowable stresses for archaic materials. Specific stresses are given for solid brick masonry walls with mortar joints filled and materials of reasonably good quality as well as for adobe, wood, steel and iron. The author has some reservations about this new code which hopefully will be resolved as more experience is gained with its use. It certainly is an important step towards codifying strength analysis of historical structures.

NEEDS FOR IMPROVED STRENGTHENING -

The previous examples illustrate several methods being used in the United States to rehabilitate old buildings. It can be seen that all of these methods rely on considerable judgment by the Engineer and little guidance is available. The economic situation of design contracts in the United States permits only limited testing of existing materials and seldom any testing to research actual material performance. For example, it has become common in California to obtain cores of brick walls and test them on their sides to evaluate the bed joint shear strength. But little work has been done to relate that test result to the actual strength of an unreinforced brick wall with openings, discontinuities, variability of mortar, and all other aspects of a real building. More needs to be known about wall failures, how brick walls really fail, what kind of anchorages are effective and how all types work. Testing must reflect dynamic loadings, not just static loadings that are easy to produce in the laboratory.

Similar research is needed in the performance of adobe walls, stone masonry, rubble concrete, straight sheathed timber walls and diaphragms, cast iron members and many other materials encountered in old buildings. Research is also desirable in evaluating the effectiveness of some strengthening procedures such as gunite applied to masonry walls, various anchorage devices, etc. All of this evaluation must consider the post-elastic performance of the materials so ductility or lack of ductility can be assessed.

Another factor somewhat removed from engineering is governmental incentives to encourage the seismic strengthening of old buildings prior to an earthquake. In the United States, there are some tax advantages for rehabilitation of old buildings. However, in many cases, the present incentive is to leave the building in its potentially hazardous state rather than spending the money necessary to provide reasonable safety to the occupants. Like most societies, it is practically and economically unfeasible to condemn all our older or somewhat hazardous buildings and prevent their continued usage until they are seismically strengthened. The process must be a gradual one, but it could use some additional appropriate governmental incentive for encouragement.

SUMMARY -

The following statements summarize the paper and its conclusions:

1. The seismic strengthening of old buildings involves considerable engineering judgment and is difficult using current Building Codes that include only modern materials and methods of construction.

2. Old buildings of types that have traditionally performed well in historic earthquakes can be strengthened by adding and strengthening members as appropriate to reduce the hazard of collapse and loss of

life in a major earthquake. Such an approach should be based on design forces less than current code as considerable uncalculated strength and redundancy exists. Cooperation of building authorities is mandatory with this approach.

3. Buildings that are seismically weak and prone to collapse in strong ground shaking should have complete, new bracing systems added in their strengthening process to provide adequate assurance of safety.

4. Building Codes dealing with rehabilitation of old structures with archaic materials, such as the California State Historical Building Code, offer the promise of guidance in rehabilitation while preserving the historic fabric of a building, but more experience is needed to evaluate the appropriateness of the provisions.

5. Research on evaluating the strength of archaic materials and strengthening techniques would be extremely beneficial to the engineering profession.

6. Government incentives to strengthen old buildings should increase the safety of the public and reduce potential hazards.

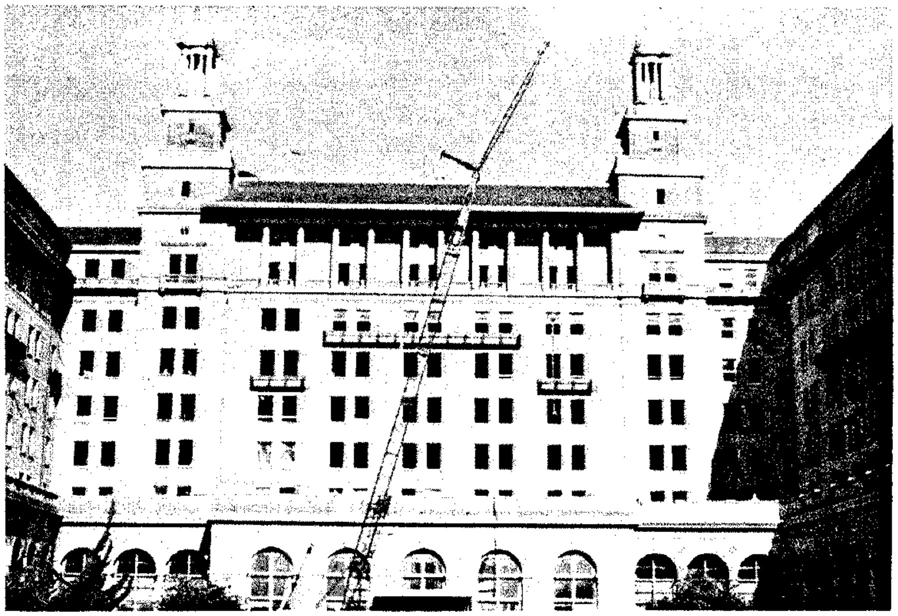

Figure 1. Front view of the Hotel Oakland of steel frame construction, concrete floors and unreinforced brick exterior walls.

Figure 2. Bracing of unreinforced brick on typical floor of Hotel Oakland restoration. Epoxy bolt in brick connected to heavy duty furring studs which spanned between floors.

Figure 3. Ground Floor of Hotel Oakland, which high story height required steel wide flange beam spanning between floors with epoxy bolts in the exterior brick walls attached for wall forces perpendicular to the wall.

REFERENCES

1. *Uniform Building Code*, International Conference of Building Officials, Whittier, California, 1979 Edition.

2. "The Effects of the San Francisco Earthquake of April 18th, 1906, on Engineering Constructions", Paper No. 1056, Transactions, American Society of Civil Engineers, Vol LIX, December 1907, pp 208-329.

3. "State Historical Building Code", California Administrative Code, Title 24, Part 8, August 24, 1979, and amended October 20, 1979.

ASEISMIC STRENGTHENING OF EXISTING REINFORCED CONCRETE BUILDINGS

by

Shunsuke Sugano *

INTRODUCTION

A number of reinforced concrete buildings damaged by recent earthquakes required extensive amounts of strengthening as well as repair for their rehabilitations [1-4]. In the case of 1968 Tokachi-oki Earthquake which significantly damaged a large number of low-rise buildings, some of the damaged buildings were strengthened by placing new walls along with the repair and they have been still in use [1,2]. The design and construction for strengthening of these buildings, however, were accomplished on the basis of experience and engineering judgement alone due to lack of appropriate guidelines. Some of the severely damaged buildings due to the latest destructive earthquake, Miyagiken-oki Earthquake of 1978, also needed strengthening. The construction, however, used various types of techniques and materials and the design was based on experimental and/or analytical investigations, or available guidelines [5-8].

While recent practices in the analytical evaluations of seismic safety of existing buildings have indicated that there has been a wide scatter in their earthquake resistances and that a considerable number of low and middle-height buildings designed and constructed on the basis of previous codes and standards may need strengthening [9-11]. It should be noted that a number of public and private buildings which were judged be hazardous were strengthened or rebuilt.

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* Chief Research Engineer, Takenaka Technical Research Laboratory, Tokyo

The aseismic strengthening of existing buildings for improvement of their earthquake resistance, thus, may be accomplished before a severe earthquake occurs or along with the repair of damaged structures. The improved resistance should be designed not only to prevent collapse but also to limit structural deflection so that architectural and mechanical elements within the building will not be severely damaged. To obtain guidelines for both design and construction of aseismic strengthening, several experimental studies have been conducted with emphasis on construction techniques and materials to strengthen existing structures [12-25]. Available test data, however, have been limited and have not been systematically reviwed because of only few years of experiences.

The importance and necessity of aseismic strengthening for rehabilitation of existing hazardous buildings have been recognized year after year in the society as well as in the engineering field, and appropriate guidelines for design and construction have been strongly required. Thus, a design guideline was proposed in 1977 by the advisory committee for the Ministry of Construction [27]. This guideline was organized to be used coupling with the evaluation method of seismic safety proposed by the same committee [26]. There have been a number of practices of strengthening based on the guideline. While experimental investigations and review of data have been needed for further informations.

The emphsis of this paper will be directed toward how our current knowledge can be used to design for increased earthquake resistance of damaged or hazardous buildings. First general design procedures will be briefly described and a brief review of experimental studies on various types of strengthening techniques will be given. Some applications of strengthening to existing buildings will be also described.

DESIGN AND CONSTRUCTION

General - The aseismic strengthening of an existing building is accomplished along the procedure shown as Fig. 1 [2]. Before the basic design and selection of construction method, detailed discussions on the results of analytical evaluation and/or field investigations of the present condition of the building are needed. Laboratory tests may be required for the detailing of construction and design calculation.

The earthquake resistance for a strengthened building should be either of the following two or their appropriate combination, as that for a new building is, that is, (1) to have sufficient lateral force capacity or (2) to have sufficient ductility as well as adequate strength. The concept of these types of earthquake resistance is schematically illustrated in Fig. 2 [27]. Thus, aims of aseismic strengthening are classified into the following three categories [20], (1) to increase the strength, (2) to increase the ductility or toughness, and (3) to balance the stiffness and/or strength of structural elements. The first category is considered be the most essential and effective for low and middle-height buildings which may require a large amount of strength so that they may resist the considerably high range of expected seismic response. Adequate strength may also be required to avoid extensive inelastic displacement even if the ductility is satisfactory. The second category is also effective when selected together with the first one if sufficiently increased strength can not be expected. It is important in the last category to eliminate the eccentricity of the stiffness distribution in a story and/or through the stories.

Construction - In accordance with the aims of strengthening, various types of techniques and materials for the construction can be selected as shown in Figs. 3 and 4 [2,19]. Generally, new elements may be placed within the existing structure to increase the strength of a total building while existing framing elements may be reinforced with new materials to improve the ductility.

In order to increase the strength there are following four types of methods [27], that is, (1) to infill walls within existing frames, (2) to brace existing frames, (3) to place wing walls with existing columns, and (4) to buttress an existing structure. For the methods (1) and (3), cast-in-place concrete or precast concrete are used and several types of connections are proposed as shown in Fig. 4. Typical details for the connections are given in Figs. 5 and 6. Careful attention must be paid to the detailing for connections because it may strongly affect the overall behavior of a strengthened structure. Attention must be also paid to casting concrete in site. Such techniques, for example, high pressure pumping, as will avoid possible gaps between new and existing concrete are recommended. In the case of bracing, though there have been few test data as well as applications, the connection should be designed with much care for the concentration of stresses in the connection.

As learned from the experiences of damage to columns during earthquakes, it is essential to improve the ductility, in another word, to increase the shear capacity of columns to increase the ductility of a total building. The following techniques, shown in Fig. 3 (2), were proposed for strengthening columns, (1) to incase the column in a steel section (circular or rectangular) grouting the gap with mortar, (2) to cover the column by steel straps welded with steel angles set at each corner of the column, and (3) to cover the column by new mortar reinforcing with welded wire fabrics. In these constructions, it is important to provide narrow gaps at the ends of a column so that the shear or bond strength may be improved without increased flexural capacity.

Design - The safety of a strengthened building may be evaluated by " Criterion on the Evaluation of Seismic Safety of Existing Reinforced Concrete Buildings"[26], or alternatively by more detailed or appropriate procedure. The evaluation of property of strengthened structures or strengthening elements may follow the previously described guideline [27], unless adequate experimental informations are available. The guideline also provides design calculation procedures for infilled walls, wing walls and reinforced columns based upon test data or theoretical approaches.

As indicated in Fig. 5 (d), the design strength of an infilled wall is given as the smaller value of (1) total shear strength of a panel and both columns, or (2) total of strengths of a column and connection along a beam plus punching shear strength of the other column, assuming that major failure occurs at the panel or at the connection. The strength of connection is determined by given empirical or theoretical equations for three types of construction techniques shown in Fig. 5 (a)-(c). The punching shear strength is also given in terms of the principal tensile stress. Flexural and shear capacities of a column with monolithically cast wing walls are obtained as the reduced values to 80 % of those of an identical monolithic column. In case of the other type of wing wall shown in Fig. 6 (c), the strength is given as that of the idealized truss system. Taking account of the thickened section and/or the strength of new steel elements, the shear capacity of a strengthened column can be evaluated. For other strengthening techniques rather than those described in the guideline, the evaluation based on experimental data is highly recommended.

RESEARCH ON ASEISMIC STRENGTHENING

As reported in references 12 to 25, various types of strengthened structures have been examined by experimental approaches during recent several years. Most of the early results were refferred as back data of the proposed design guideline for aseismic strengthening.

The earliest test data were of improved ductility of columns reinforced by the techniques shown in Fig. 3(2) (Kokusho [12] and Sasaki [22]), and of improved lateral force capacity of columns and frames strengthened by adding cast-in-place or precast concrete walls (Higashi [12,16]. Afterwards, one-story infilled walls of cast-in-place concrete having different detailings of connections were tested by Kokusho and others [12-14]. Various types of bracing as well as infilling techniques were reported by Yamaguchi and Sugano [18-20] and by Higashi [17]. The technique to improve the ductility of columns by using tie plates was recently discussed by Arakawa [23]. Typical test programs and results are briefly illustrated in Fig. 7 through 11 with emphsis on different types of strengthening methods.

Infilled Concrete Walls - A series of tests by Kokusho for three types of infilled walls shown in Fig. 7 indicate that (1) infilled walls provide reasonable strength, however, dowels may simultaneously fail in shear at their screw parts, (2) it is effective to provide gaps along columns so that walls may behave in a ductile manner, and (3) chipped shear key may provide as much strength as that of monolithic wall. While it is indicated on the basis of the test by Yamaguchi and Sugano (Fig. 9) that infilled wall may have as much strength as that of a monolithic wall when adequate connections are provided, in addition, it is preferable to arrange connections all around the frame. The latest test by Higashi reports that multiple precast walls promise sufficient ductility, as indicated by Hanson [24,25], though the strength is much less than those of cast-in-place concrete walls.

After a brief review of available test data [13,15,17,20,28], Sugano proposed the following guidelines from the viewpoint of increased strength of infilled walls with dowel connections (Fig.9(e), [20]), (1) when the required strength is more than 60% of that of a monolithic wall (Qw) or $2.0\sqrt{Fc}$ kg/cm^2 in terms of the nominal shear stress, where Fc is concrete strength, dowels should be designed to have the strength more than 10 kg/cm^2, (2) a wall may have the strength more than 0.4Qw or $1.0\sqrt{Fc}$ even without any connection, and (3) a wall without connections along columns may have the strength as much as 0.6Qw or $2.0\sqrt{Fc}$.

Braced Frames - Available test data of braced frame are very limited. Higashi selected K and ◇-shaped braces of steel while Yamaguchi and Sugano selected two x-braces of steel for compression and tension, respectively. The test results of four braced frames indicate that the bracing technique promises moderate strength as well as adequate ductility and/or energy absorption (Fig. 9(b)). Attention should be carefully paid to the detailing of connections since it may strongly affect the overall hysteretic behavior of braced frames.

Additional Wall Construction - Higashi and Okubo strengthened columns or frames by placing wing wall with a column or by placing wall panels within a frame, respectively (Fig. 8). The type of additional wall were examined. The findings are (1) cast-in-place concrete wing walls provide almost identical strength to that of a monolithic column while precast walls provide less strength but larger displacement ability, (2) precast panels also have advantage of sufficient ductility in a frame and (3) precast panels may be idealized into a truss element for the strength calculation.

Reinforced Columns - Three types of strengthening techniques corresponding to each of those in Fig. 3(2) were discussed by Sasaki (Fig. 10), Arakawa [23] and Kokusho (Fig. 11), respectively. As indicated in Fig. 10(c), the effect of steel incasements and straps on improving the both strength and ductility of weak columns was significant. Also the thickening technique with welded wire fabrics significantly improved the ductility of poor columns (Fig. 11(c),(d)). Arakawa reports that tie plates are also effective to improve the ductility of columns since they prevent shear failure and delay the crush of concrete.

Effect of Strengthening - Although available test data have been very limited, the outline of hysteresis curves of strengthened structures by various types of techniques are schematically illustrated by Sugano in Fig. 9(c). Note that the figure provides only ideas how much strength and displacement we may have by using available techniques. As indicated in the figure, infilled walls may have more than 0.6 or 3.5 times the strength of a monolithic wall or unstrengthened frame, respectively, when adequate connection is provided. Steel elements provide less increased capacity than those of concrete walls, however, they promise larger ductility. Strengthened columns by wing walls may have up to two times the strength of the original column and the displacement at the load capacity is more than 0.015 radian. Generally, the smaller the increased capacity, the larger is the displacement ability.

STRENGTHENING FOR DAMAGED BUILDINGS

Three cases of strengthenings for damaged buildings due to recent destructive earthquakes are shown in Figs. 12 to 14, and the outline of the construction is briefly described as follows.

School Building (1)[6,7] - A five-story college building shown in Fig. 12 (a) and (b) suffered severe damage to captive columns at the northside as shown in Fig. 12(c) during the Miyagikenoki Earthquake of June, 1978. Severely damaged columns were replaced with new concrete providing additional reinforcements. The transverse direction was strengthened through the stories by infilled walls. The existing walls were thikened. The longitudinal direction was also reinforced through the stories with diagonally configulated steel braces as shown in Fig. 12 (d). Braces were connected with the exterior side of existing beams by means of steel platforms. An experimental program was undertaken to investigate the behavior of braces. Spandrel walls were drilled along columns to reduce their contribution to columns. The microtremor measurement after the construction indicated that the building recovered the stiffness almost identical to that before the earthquake.

School Building (2)[5] - Three buildings in the school sustained severe or moderate damage, as shown in Fig. 13(a) and (b), mainly to captive columns at the north side during the earthquake of June, 1978. The damage oriented to the longitudinal direction. Severely damaged columns were replaced with new concrete providing additional reinforcements. Some of other columns were only epoxied or replaced at cover concrete. In each building, concrete walls were placed within northside longitudinal frames at every two spans in the lowest two stories and four spans for the top story (Fig. 13(c)). Epoxied dowels were used for the connection. The increased lateral force capacity was estimated be almost twice of that before the earthquake.

City Hall [7,31] - A two-story simple structure shown in Fig. 14(a) was under construction for aseismic strengthening as shown in Fig. 14(b) when the earthquake of June, 1978 also hit this area, because most the columns of the first story sustained moderate or severe damage due to the earlier earthquake of February of the same year (Fig. 14(d)). The damage by the later earthquake was only minor despite the ground shaking was supposed to be much more stronger than that of the earlier earthquake. Apparently, the constructed walls in Fig. 14(b) significantly contributed to the earthquake resistance of the total building. Thus, an encouraging observation, that the strengthened structure sustained only minor damage during a severe earthquake, was made. It should be noted that this was one of very few experiences of strengthened structures.

CONCLUDING REMARKS

There have remained many problems to be solved in the fields of design, construction and research for aseismic strenghening because available data and our experiences have been limited. For further informations in these fields, the following items should be discussed or practiced. (1) The effect of workmanship and detailing for connections on the overall behavior of strengthened structures. (2) Evaluation and review of existing data. (3) The effect of strengthened structure over the behavior of a total building. and (4) Collection of construction records.

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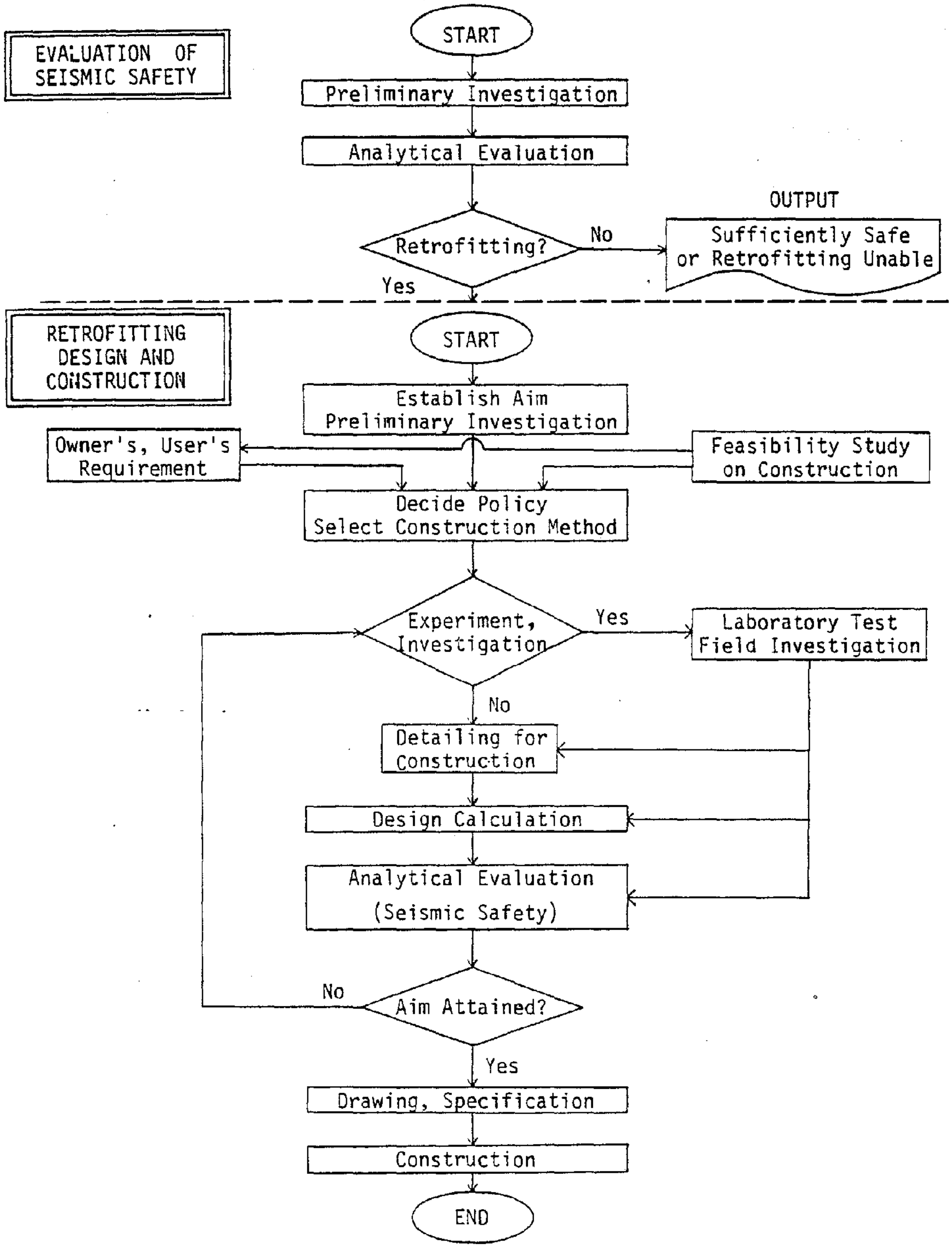
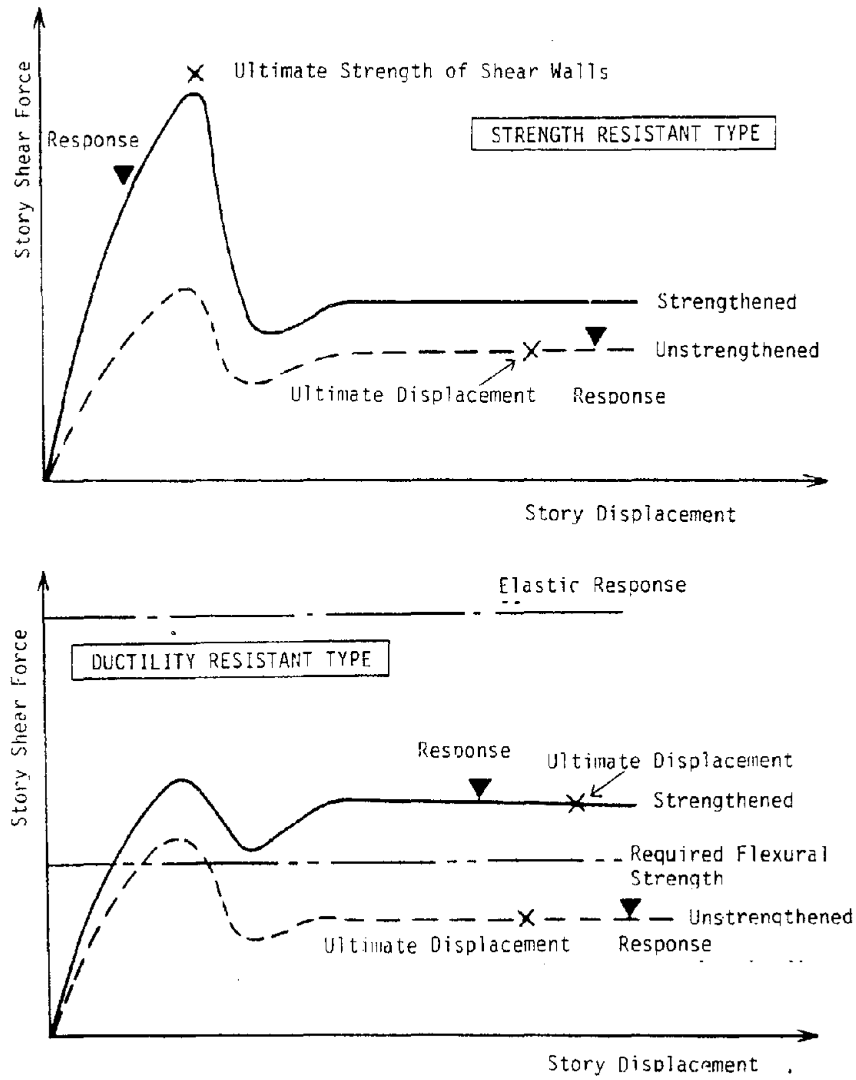
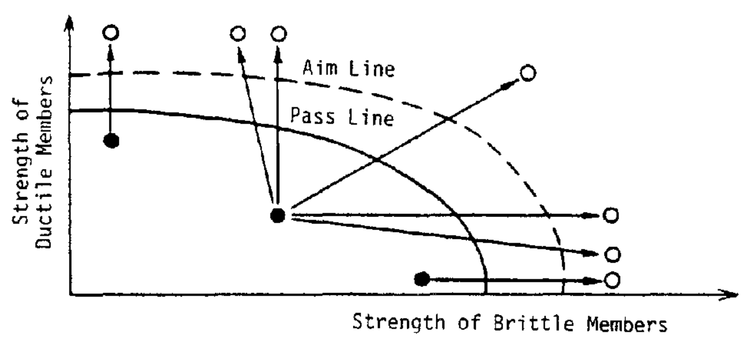


FIG. 1 FLOW CHART OF DESIGN AND CONSTRUCTION OF ASEISMIC STRENGTHENING(2)

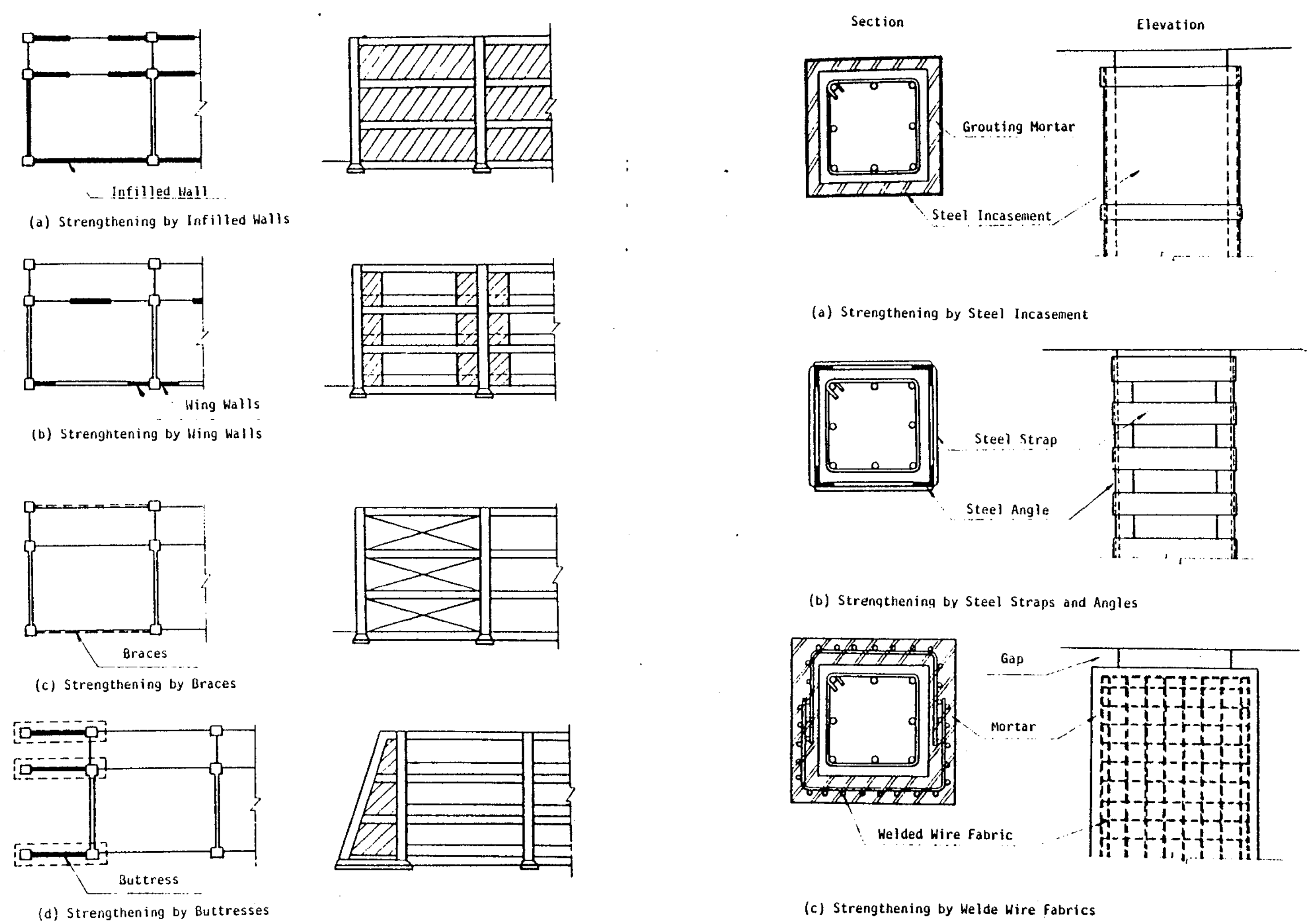


(a) Type of Earthquake Resistance



(b) Aim of Strengthening

FIG. 2 AIM OF ASEISMIC STRENGTHENING (27)



(1) Techniaues to Increase Strength

(2) Techniques to Increase Ductility

FIG. 3 CONSTRUCTION TECHNIQUES FOR STRENGTHENING (2)

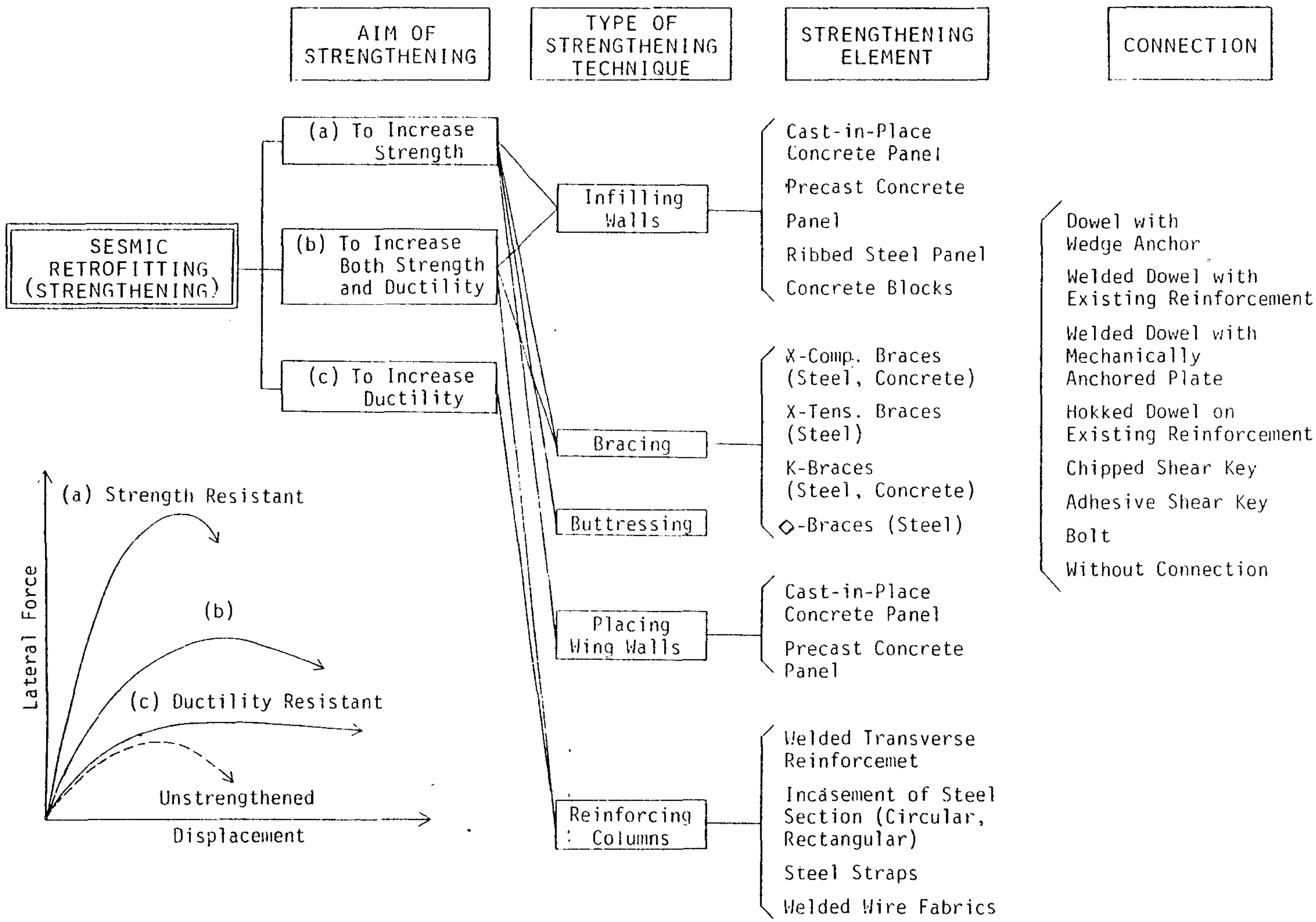


FIG. 4 TYPICAL ASEISMIC STRENGTHENING METHOD (19)

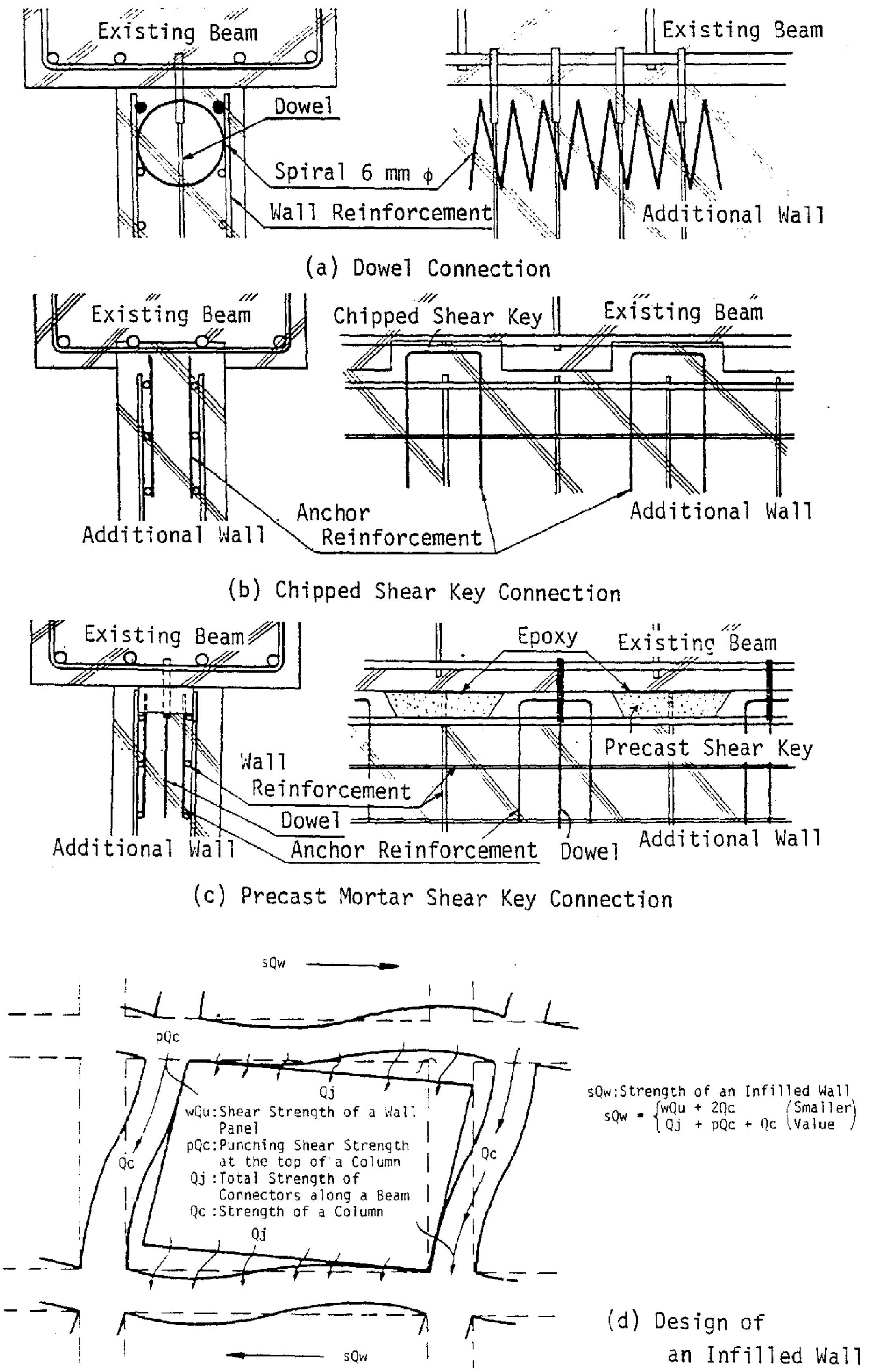
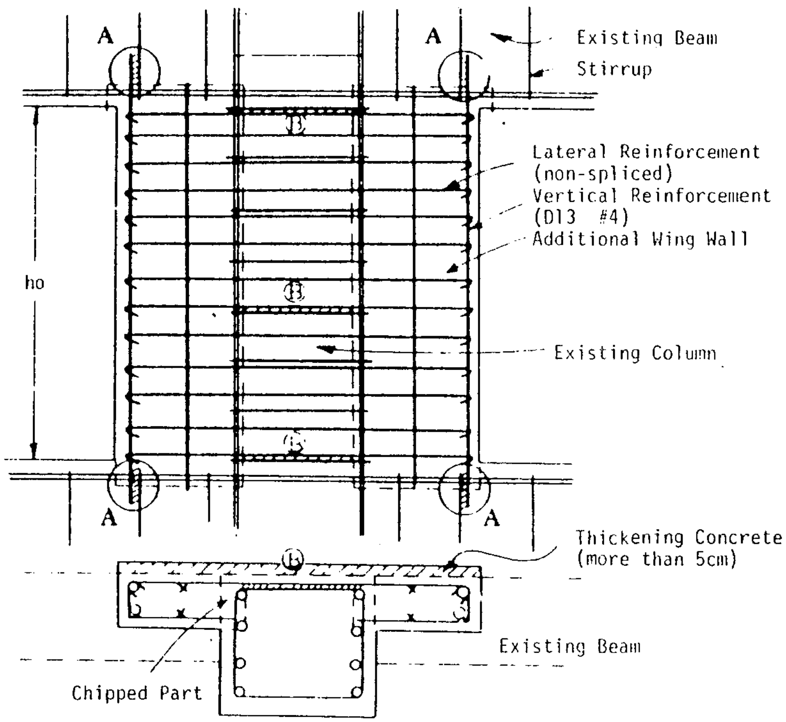
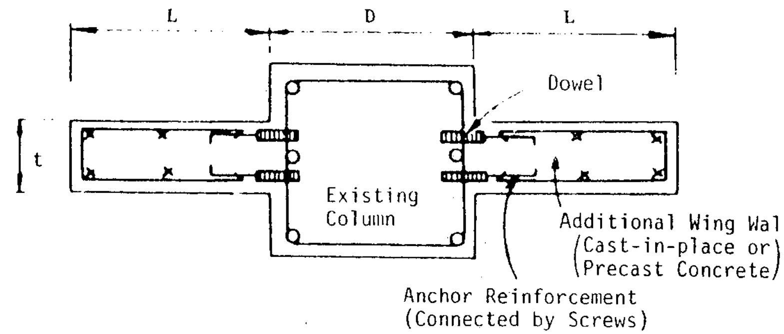


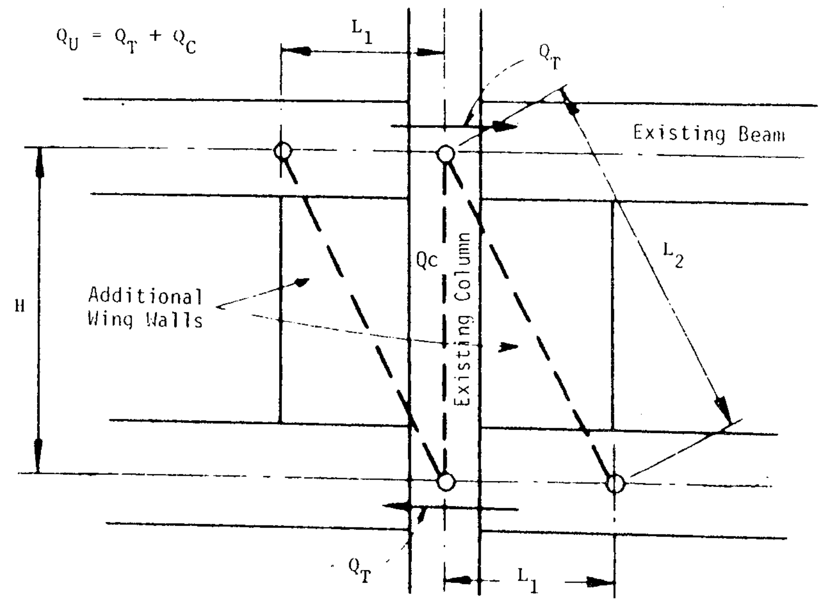
FIG. 5 DESIGN AND CONSTRUCTION OF INFILLED WALLS (27)



(a) Monolithically Cast Wing Walls

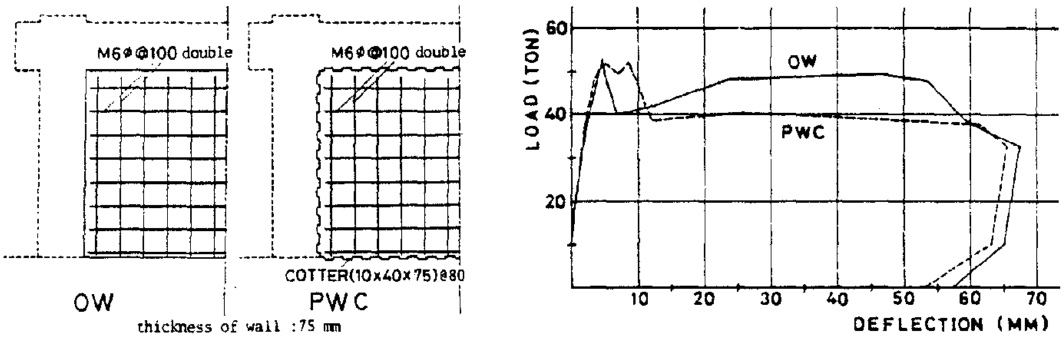


(b) Wing Walls with Dowel Connections

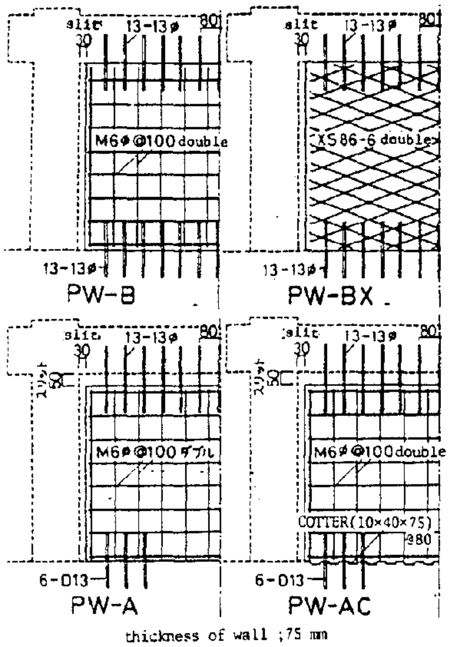


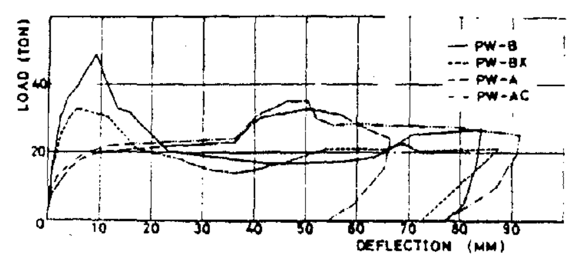
(c) Design of Wing Walls with Dowel Connection

FIG. 6 DESIGN AND CONSTRUCTION OF WING WALLS (27)

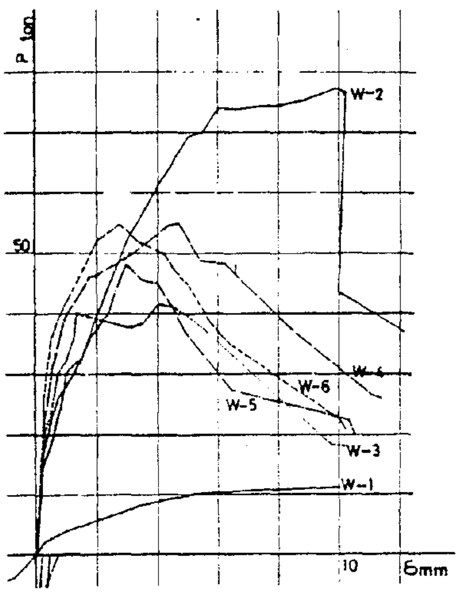


(a) Test Specimens and Results (1)





(b) Test Specimens and Results (2)

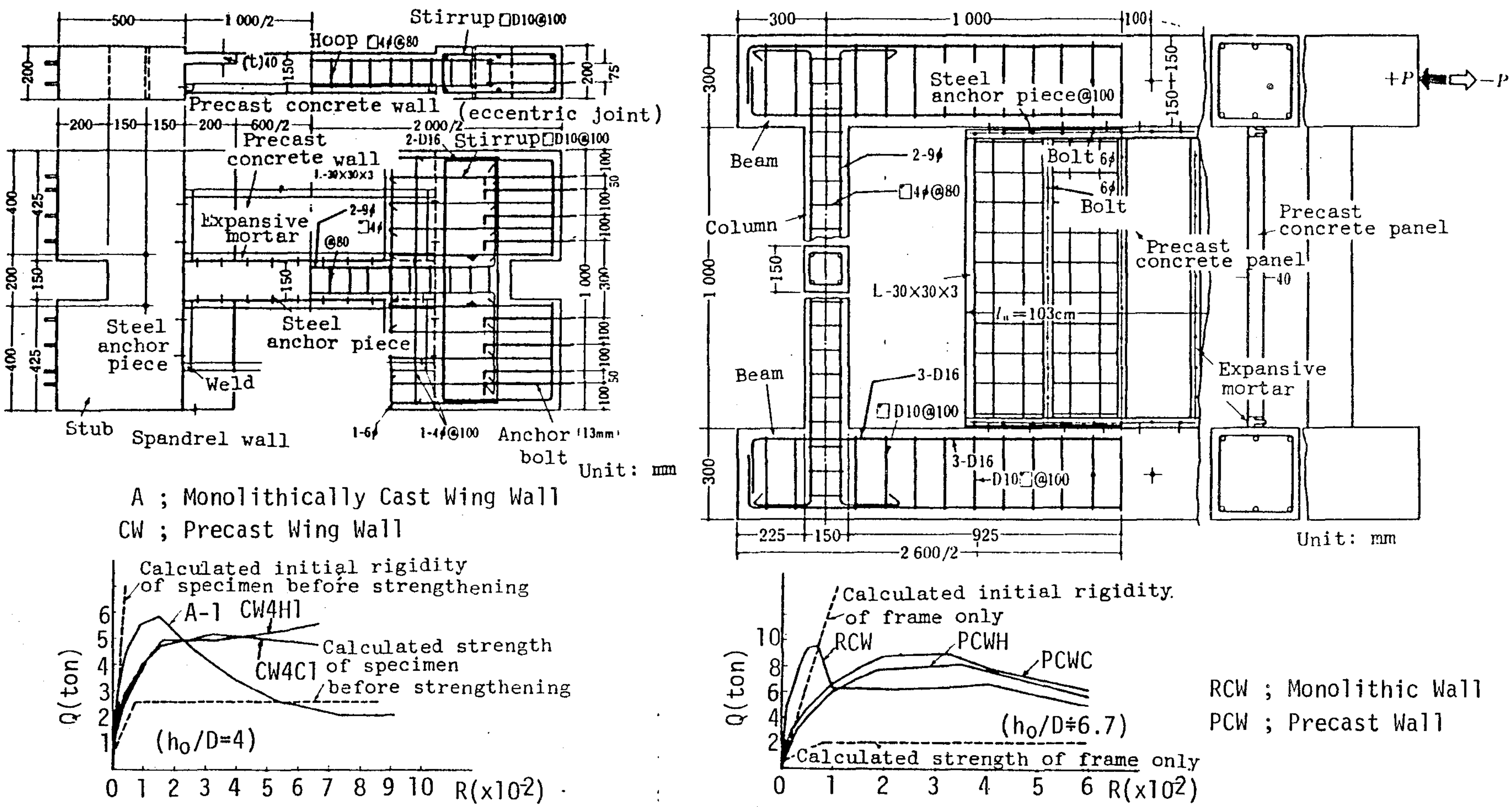


W-1 : Unstrengthened Frame
W-2 : Monolithic Wall
W-3 : Epoxied Mortar Shear Key
W-4 : Dowel with Wedge Anchor, along Top Beam, Roughing
W-5 : Dowel all around the Frame
W-6 : Dowel all around the Frame Roughing

(c) Test Results (3)

FIG. 7 TESTS OF INFILLED WALLS

(After Kokusho [12-14])



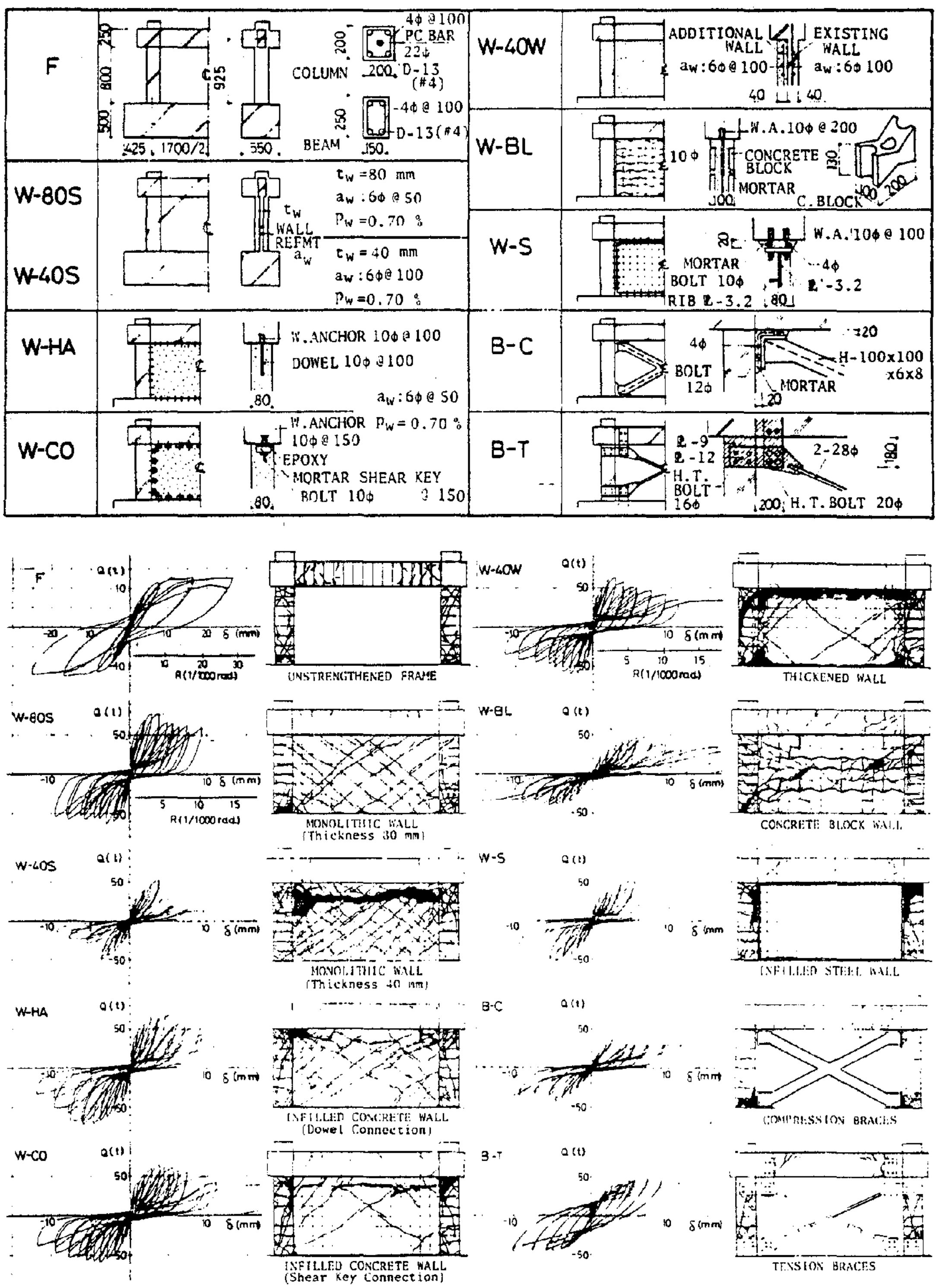
(a) Test Specimens and Results of Wing Walls

(b) Test Specimens and Results of Strengthened Frames

FIG. 8 TESTS OF WING WALLS AND STRENGTHENED FRAMES (After Higashi [12,16])

(a) TEST SPECIMENS

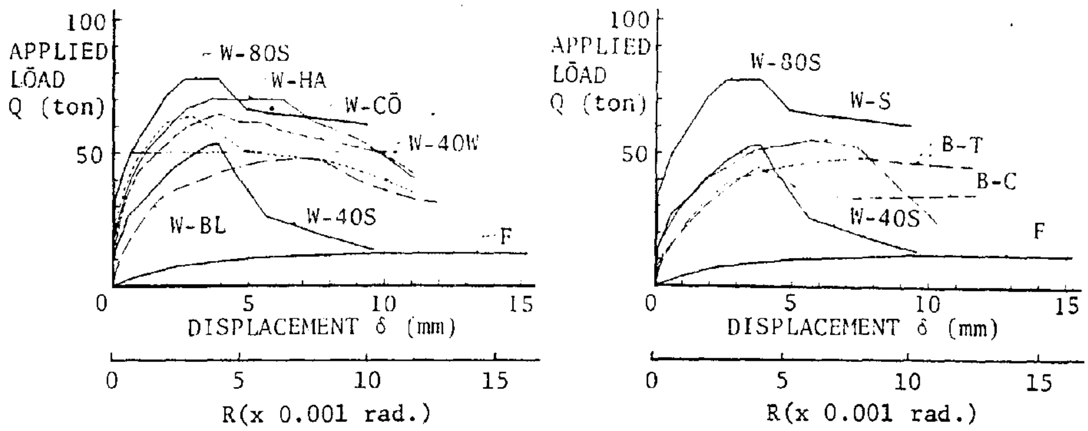
unit in mm



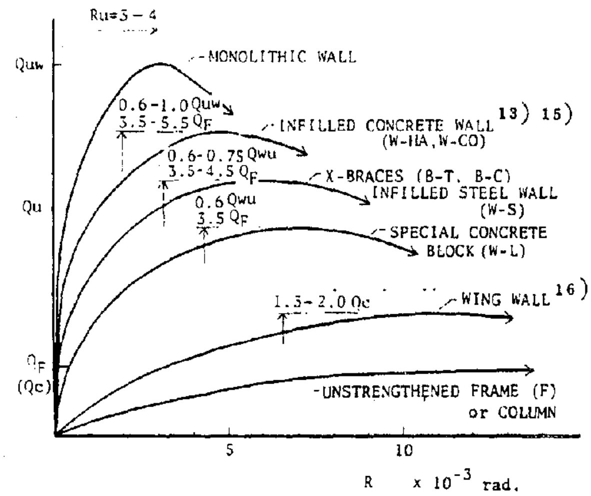
(b) HYSTERESIS CURVES AND FAILURE PATTERNS

FIG. 9 TESTS OF INFILLED WALLS AND BRACED FRAMES

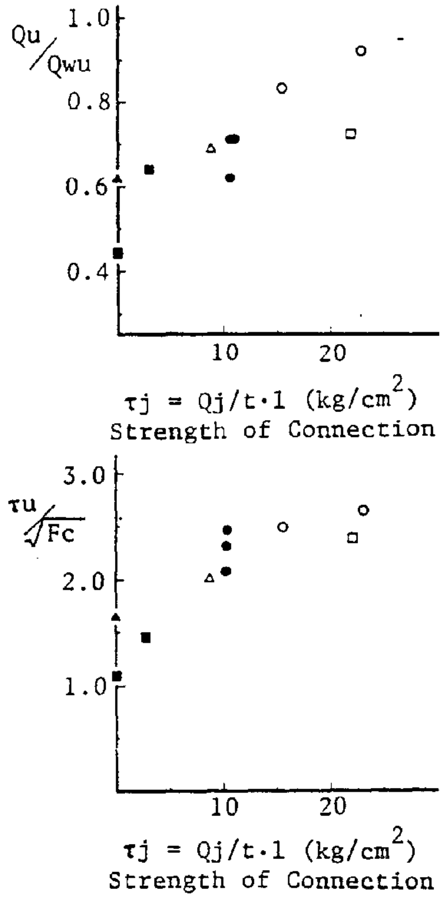
(After Yamaguchi and Sugano[18-20])

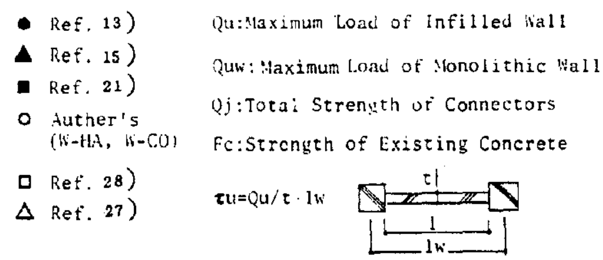


(c) ENVELOPES OF HYSTERESIS CURVES



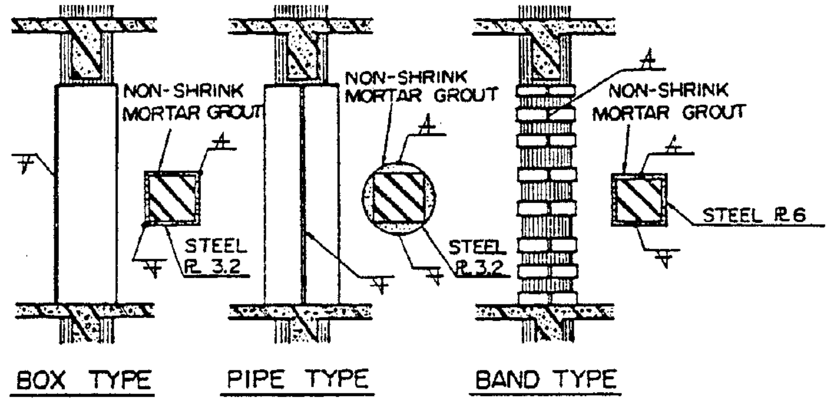
(d) OUTLINE OF LOAD vs DISPLACEMENT FOR DIFFERENT STRENGTHENING TECHNIQUES



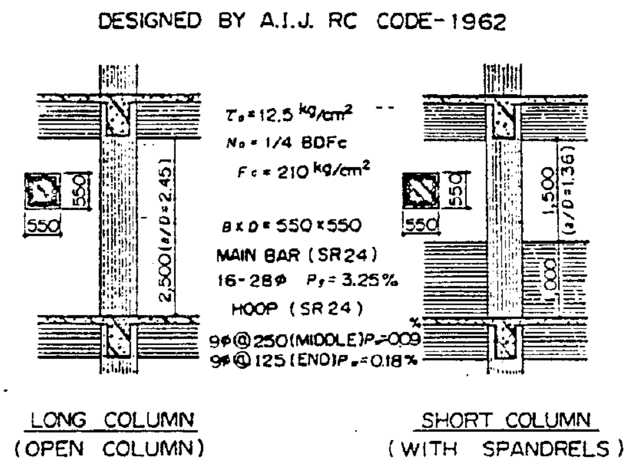


(e) STRENGTH OF INFILLED WALL vs STRENGTH OF CONNECTION

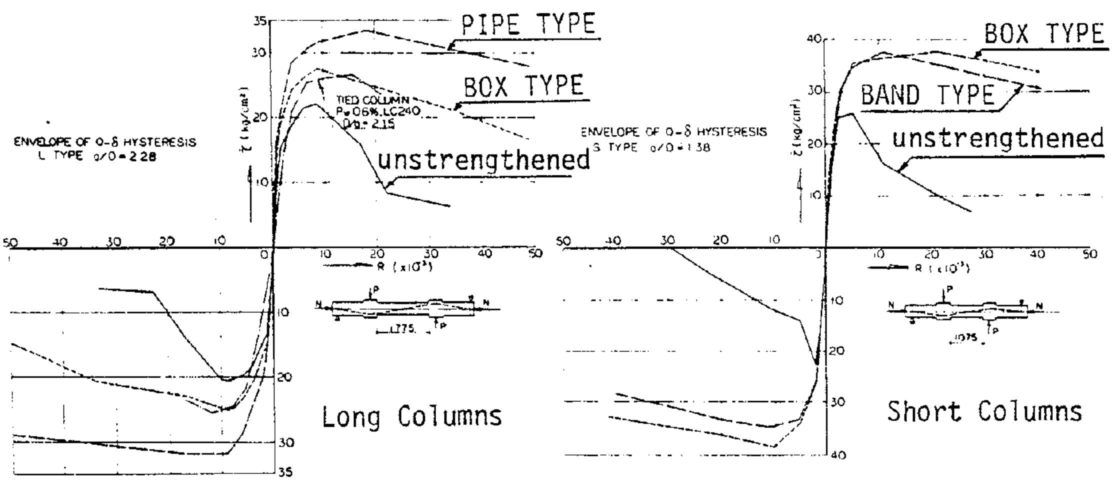
FIG. 9 (Continued)



(a) Strengthening Method

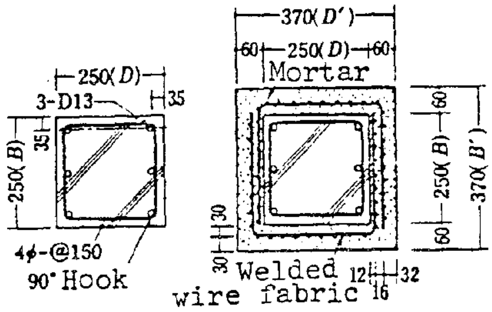


(b) Unstrengthened Columns

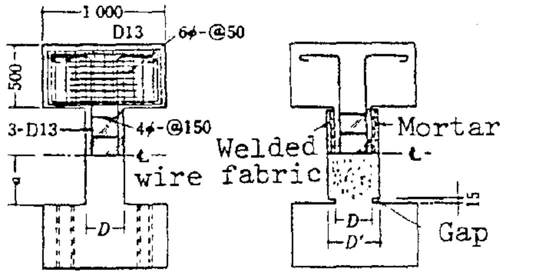


(c) Test Results

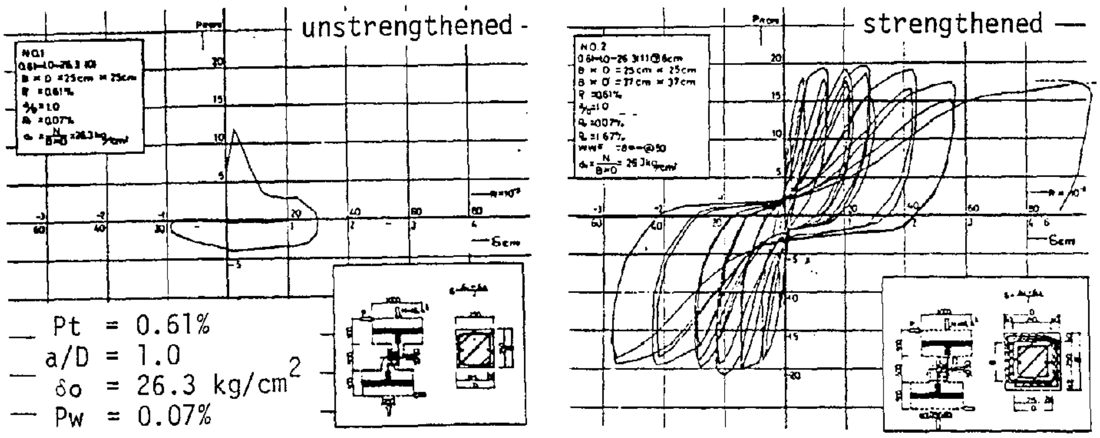
FIG. 10 TESTS OF STRENGTHENED COLUMNS (After Sasaki)[22]



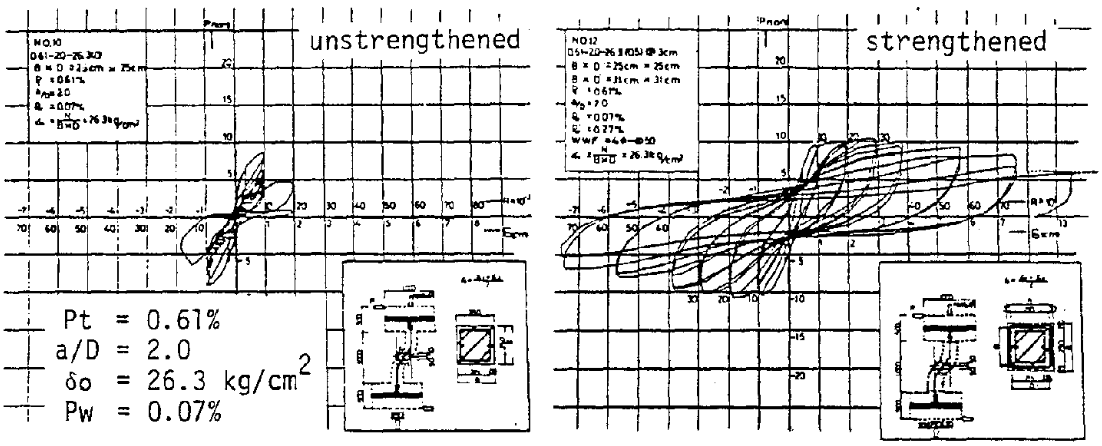
(a) Strengthening Method



(b) Test Specimen

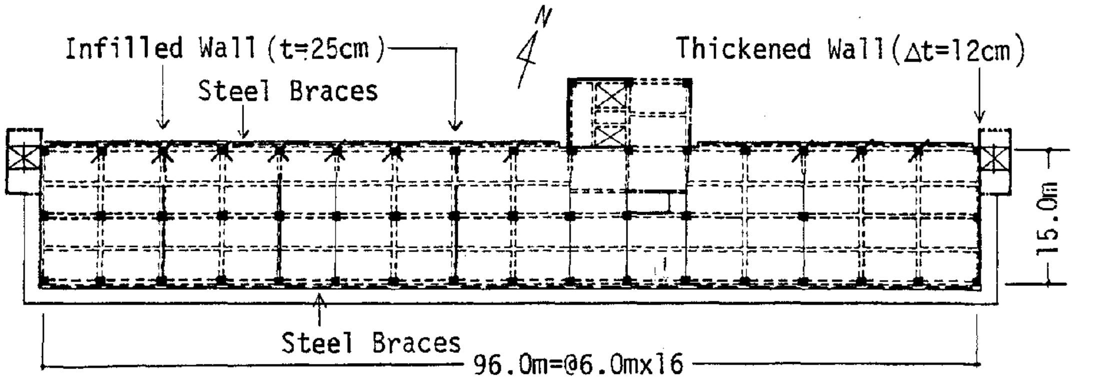


(c) Test Result of Short Column

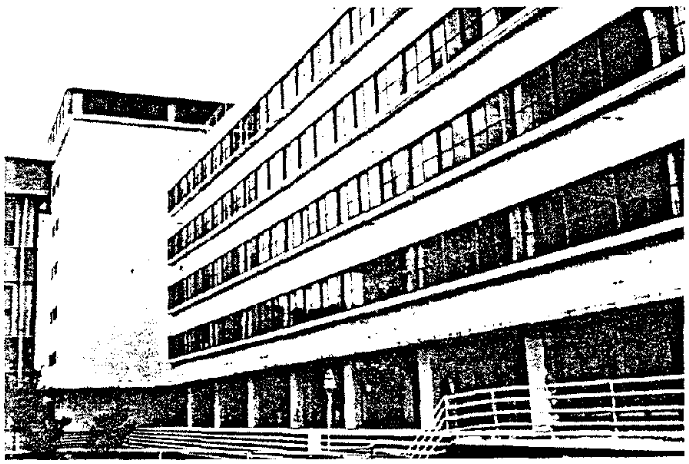


(d) Test Result of Long Column

FIG. 11 TESTS OF STRENGTHENED COLUMNS (After Kokusho [15])

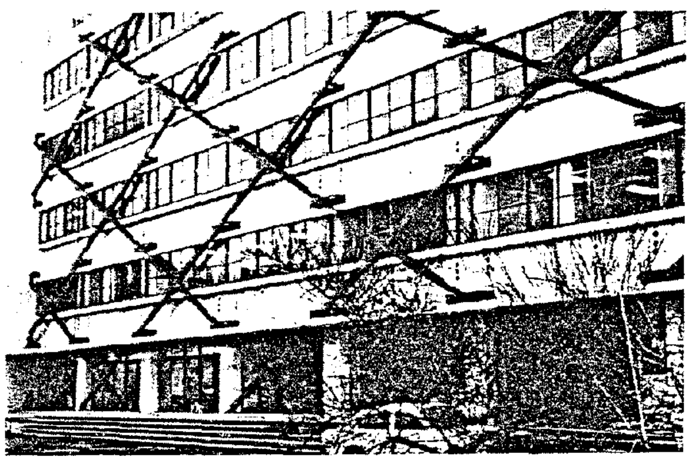


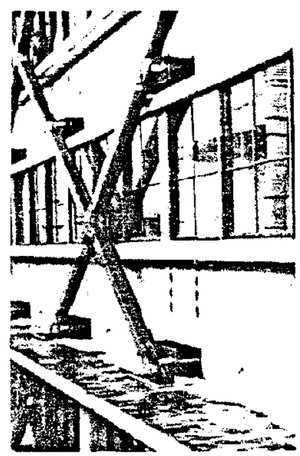
(a) Typical Floor Plan(6,7)



(b) North Facade

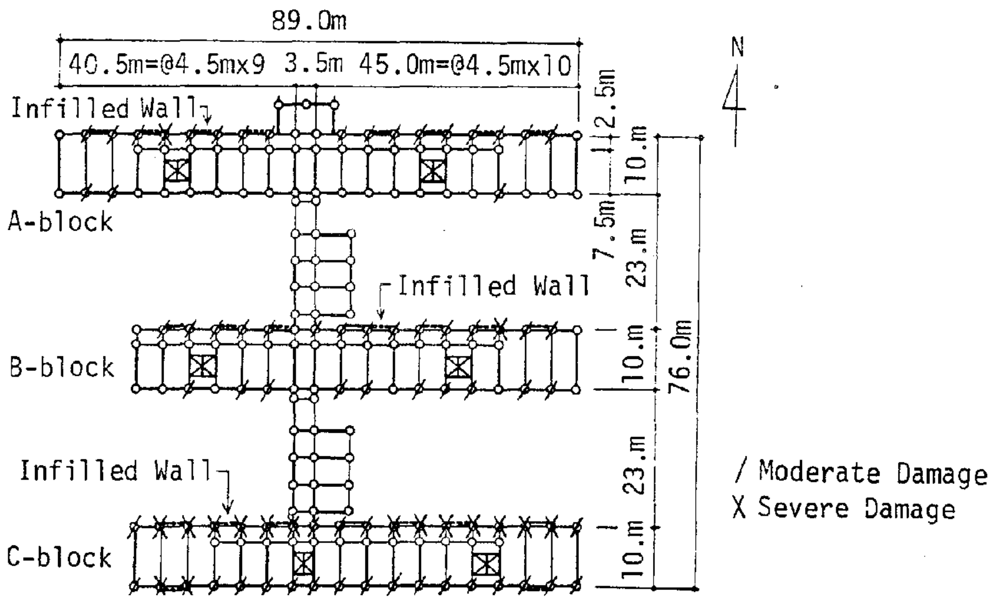
(c) Damage to a Column (June, 1978)



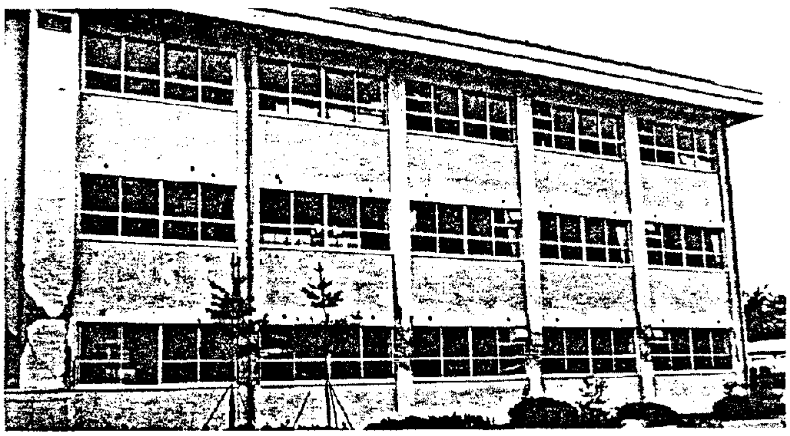


(d) After Strengthening

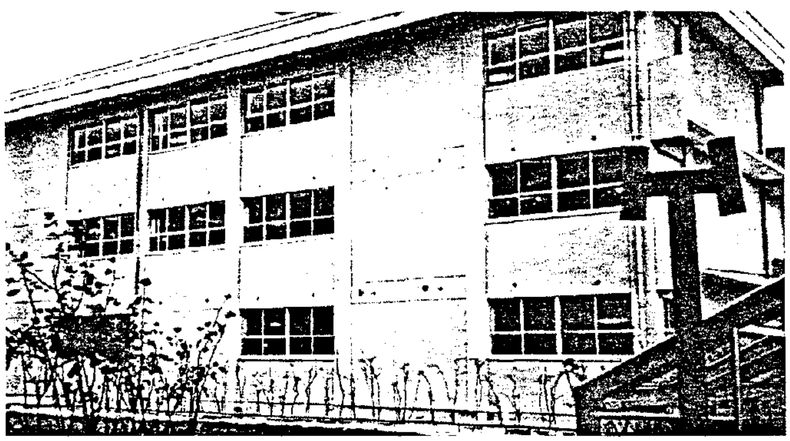
FIG. 12 A STRENGTHENED SCHOOL BUILDING



(a) Typical Floor Plan(5)

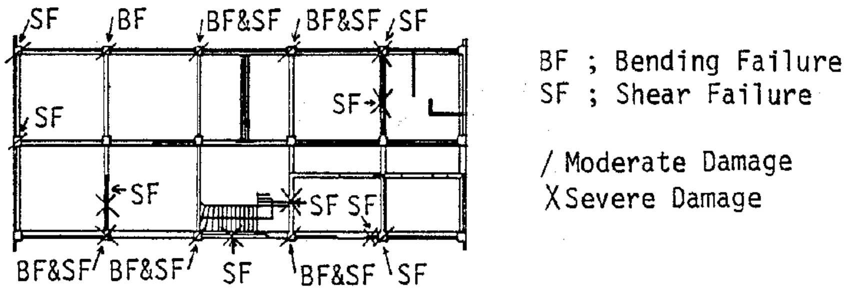


(b) Damage to Columns (June, 1978)

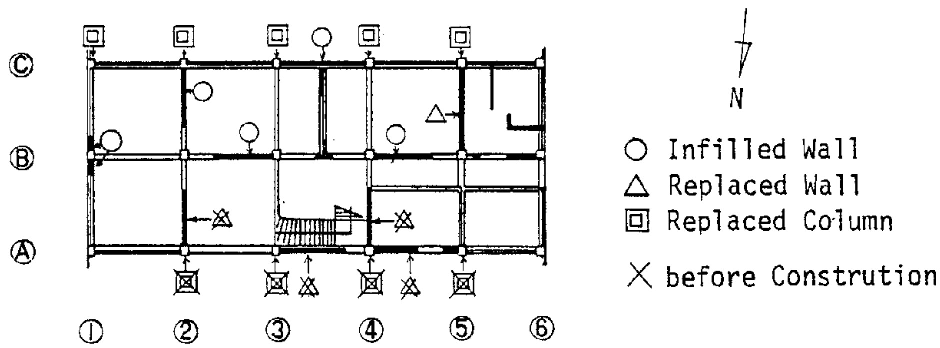


(c) After Strengthening

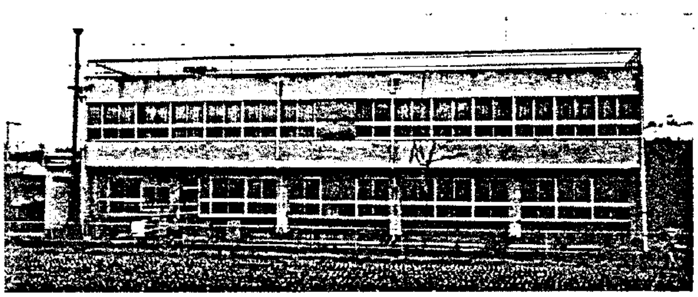
FIG. 13 A STRENGTHENED SCHOOL BUILDING



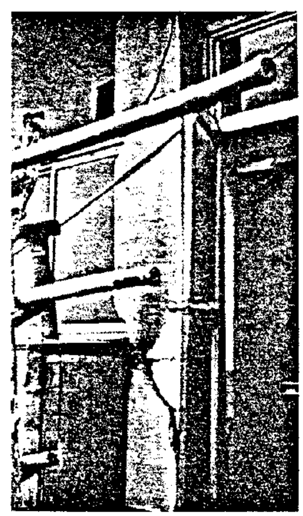
(a) Floor Plan and Damage to Members (Feb., 1978)[7]



(b) Construction for Strengthening (June, 1978)[7]



(c) South Facade (Feb., 1978)



(d) Damage to a Column
(Feb., 1978)

(e) Construction for Strenghtening (June, 1978)

FIG. 14 A STRENGTHENED CITY HALL

REPAIR AND RETROFIT OF BUILDINGS

OVERVIEW OF U.S. EXPERIENCES -- CURRENT PRACTICE AND WEAKNESSES

by

James Warner
Consulting Engineer
La Canada, California

INTRODUCTION

The present state of the art for repairing and strengthening existing structures employs methods which have largely been developed through experience and thus are empirical in nature. Because of past limited need for such work, the existence of well established standards or firms which specialize therein and thus maintain the capability to design, develop, test, and apply optimum remedial procedures is limited. Following every major earthquake, however, vast numbers of "over-night experts" seem to appear. Accordingly, due to the infrequent requirement for seismic damage repair in any given area and lack of guide codes or recommended procedures, owners, their engineering consultants, and the controlling authorities are often restricted in utilization of the most optimal methods, and less than desirable results are often obtained.

In the case of seismic damage repair, the exact requirements or objectives of a given program are often quite obvious, i.e., those portions of the structure needing repair have been clearly defined by having failed or received significant damage. In the case of strengthening of existing buildings however, the engineer must depend upon inspection, analysis, and to a very large degree, engineering judgment to determine the areas of weakness that are to receive attention. In either case, existing building codes, in general, do not address themselves toward remedial work, though often requiring any such work to upgrade the particular structure to full code compliance. This frequently results in employment of other than optimal remedial procedures. Thus, present practice is generally restricted to employment of established methods which are, at least to some degree, covered by existing codes. Such restrictions very often limit the ability of the engineer and constructor in effecting optimal as well as economical retrofitting.

CURRENT PRACTICE

Present practice generally involves strengthening of existing elements, addition of new force resisting elements, or a combination of the two. In addition, the anchorage of non-structural elements (wall claddings, ornamental components, etc.) is of prime importance.

Strengthening Existing Elements

Existing elements are generally improved by increasing their cohesive nature through injection of grout or other structural adhesive, containing their mass by encasement, increasing their dimension by addition of section, or a combination of the above. Occasionally they may be braced by the addition of ties, struts, or other connecting elements. Shear walls are often improved by the addition of section. Filling in of existing openings is also a frequent expedient. Strengthening of roof and floor diaphragms usually involves increasing their thickness or the addition of stiffening ribs. Foundation elements are improved by increasing their plan dimensions, extending their depth or both. In addition, underlying soil is sometimes stabilized.

New Elements

In addition to strengthening existing shear walls, new shear walls are frequently constructed. Such new walls often replace existing interior walls which in older buildings are frequently of a non-structural nature. Load transfer to such new walls is generally through existing strengthened or in some cases new floor and roof diaphragms. Where required the addition of new drag members to transfer lateral forces to the shear walls is frequently made.

Crack Repair

Perhaps the number-one consideration in any remedial treatment is repair of existing cracks. The use of pressure-injected low-viscosity epoxy resin has become a fairly standard practice over the last decade or so. In practice the cracks are first sealed in order to contain the injected resin. The preferred sealing material is a thixotropic epoxy; however, both thermo-setting wax and cementious sealing materials have been utilized. Provision for injection is generally provided on a spacing slightly greater than the thickness of the member being repaired. The preferred method involves the use of preformed plastic injection ports with appropriate stoppers (normally standard corks). Another method commonly used, however, involves the placement of a 6 mm (1/4 in.) wide piece of masking tape over the crack at proposed injection locations prior to sealing. Before the sealing material has hardened, the tape is removed, leaving that portion of the crack exposed. Injection is then made utilizing a rubber ring gasket on the injection nozzle which is held tightly against the open crack to prevent leakage. The open crack is then sealed with a paraffin wax material following injection.

Two basic injection methods are commonly practiced. One involves automated proportioning pump-in-head mixing equipment, the other batch mixing followed by injection from a pressurized vessel. Although there remains some controversy as to the best method of application, experience has indicated that the in-line mixing system has questionable results when injection of very fine cracks (less than .12mm [.005 in] is involved although where applicable it is faster and somewhat more economical. Pressure pots have the disadvantage of tending to hold the exotherm heat with subsequent premature setting of the material. The use of refrigerated pots largely overcomes this limitation, however. [1] Because there are wide variations in the properties and proportions of different low-viscosity epoxy systems, it is important to match the equipment to the specific formulation when utilizing in-head mixing equipment. Likewise, the properties of the material must be considered and matched to the individual job requirement regardless of the method of injection.

Complete and proper injection requires sealing and installation of ports on both sides of the member being injected. Injection is started at the lowest port on one side and continued until resin appears at the next higher port. The injection nozzle is then moved to the next port and the process repeated. Injection ports are sealed as soon as the injection head is removed from them. Likewise the "inspection" or "vent" ports on the opposite side of the member are sealed as the material appears in them. Complete filling of the crack is assured by appearance of the epoxy material at <u>all</u> port locations. The injection phase is, therefore, a two man operation requiring one man on each side of the member. In most instances, a two-way telephone system is required to facilitate proper communication.

As aforementioned, in order for the epoxy injection to be effective, it is imperative that the cracks be free of dirt, grease, or other contaminants. In relatively new cracks resulting from recent seismic events, satisfactory cleaning can usually be accomplished by vacuuming ahead of the sealing operation. In older cracks special methods including flushing with water or solvents may be required. When flushing materials other than water are used, it is extremely important to confirm their compatibility with the existing concrete as well as the epoxy resin to be used. The use of acids for this purpose has been reported, however, the advisability of such use is questionable, as even with thorough flushing, residual acids may remain. Even minute residues thereof can result in serious corrosion damage to the reinforcing steel. Water blasting has been suggested as a cleaning aid as has blowing the cracks with compressed air. Except in the case of relatively wide cracks [6 mm (1/4 in)] and greater, the practice should be discouraged due to the tendency to drive the contaminant farther into the crack. Successful crack repair cannot be made with epoxy resins unless the crack surfaces are clean. Such repair should not be considered for old cracks which are contaminated to a degree that precludes proper cleaning. Where cracks are subject to moisture, the epoxy material used must be compatible with such conditions. Epoxy resins are generally limited to use on cracks with a maximum width of approximately 6 mm (1/4 in). They can be injected in cracks as small as .025 mm (.001 in) or less, however, .10 mm (.004 in) is a more practical lower limit.

<u>Spall Repair</u>

Relatively minor spalls are routinely repaired by shotcrete, epoxy-sand mortar, non-shrink cementious grouts, or standard cement-sand mortar or dry-pack. Where non-shrink grout or cement sand mortars are used, bonding agents of moisture compatible epoxy, polymer emulsion, or neat cement-water paste are sometimes used. It is important that all loose material be removed from such areas and the surface properly roughened and free of contaminants prior to patching.

<u>Shattered Concrete Replacement</u>

Where badly fractured or shattered concrete exists, complete removal and replacement is generally preferred. Reinforcing steel which has been unduly stressed will require correction as hereinafter detailed. Concrete replacement is usually made with shotcrete, preplaced aggregate concrete, or standard portland cement concrete. Type K (shrinkage compensating) cements are frequently used in such applications.

Filling Non-Visible Voids

Non-visible voids such as rock pockets, honeycomb, or excessive porosity within concrete members, or unfilled joints or cells within masonry infill panels, are frequently filled in strengthening applications. In practice, small diameter holes (approximately 2.5 cm [1 in]) are drilled with sufficient frequency to intercept the voids. The extent and configuration of the voiding can often be established by the injection of compressed air or water into the holes, combined with appropriate monitoring of return locations. In the case of relatively minor voids in concrete, epoxy resin or expansive cement grout has been used. In such instances where the void spaces are small, the cementious mixture generally consists of neat portland cement, water, and an expansive admixture, and is injected in a relatively fluid consistency. Polymer type additives are sometimes incorporated in order to increase bond strength. Such mixtures may also contain very fine sand in a proportion of from 1/2 to 1-1/2 times the cement. Flyash or natural pozzolan is sometimes used to replace up to 50% of the cement.

In the case of larger voids, expansive cement grout or epoxy-ceramic foam is used. Expansive cement grouts used in such instances are similar to those used for minor voids except that they may contain sand up to approximately four times the proportions of cementing material and are generally of a thicker consistency ranging to heavy, mortar-like where large voids are involved. Cement grouts used for such purposes have the advantage of similarity with the substrate materials, and relatively low material cost. Principal disadvantages are the relative high weight and somewhat messier injection requirements. Proper injection of cement grouts requires prewetting of the substrate by injection of water. Accordingly, the excess water must be disposed of and the repaired element will be damp for an extended time period. Such conditions will affect the existing finishes on the element and may render the procedure unsatisfactory in the case of occupied structures. Epoxy-ceramic foams have the advantage of relatively light weight, very high bond strength, and relative ease in controlling placement limits and leakage, due to their generally rapid foaming and set periods. The principal disadvantages are high material costs and relatively low compressive strength. Because of their highly expansive nature (expansion as great as 20 times their original volume) and their high bond strength, such materials have proven extremely useful in the reinforcement and bonding of masonry infill panels especially where the bonding of wall surfacing materials is required.

Bolting, Strapping and Bracing

The continuity between elements is sometimes improved by direct bolting or the placement of steel straps bolted in place across joints or cracks. [8] Parapets, towers, overhanging cornices and similar members are frequently braced by structural steel members which are bolted in place or secured by embedment in replacement mortar concrete or resinous material. [2,8,9] Where bolting through existing concrete is used, effectiveness can be greatly increased by filling any remaining space between the bolt and the hole with epoxy material.

Increasing Section of Existing and Provision of New Elements

Regardless of the particular material or method used for increasing section or provision of new elements, careful consideration must be given to provide for uniform distribution of stress from the new or strengthened elements or assemblies, to the remainder of the existing structure. Special attention should be directed toward tying the floor and roof diaphragms into the lateral force-resisting system.

Shear transfer -- Provision for shear transfer and bond development must receive adequate consideration and care during construction. In general, all existing concrete surfaces that are to be joined to new concrete should be sandblasted or chipped to a clean, rough condition providing significant exposure of the aggregate. In joints which will be subject to high shear, additional roughening with pointed chipping tools to an amplitude of 6mm (1/4 in) is a frequent requirement. [10,11] In many cases the chipping of keyways may be required. [2] Additional shear resistance can be achieved through the installation of powder-driven pins, wedge-type anchors and grouted rebar dowels. Where the replacement material is shotcrete or preplaced aggregate concrete, the use of bond coating is not recommended and, in fact, carefully controlled field tests [3] have indicated the use of such actually results in a deleterious effect when used in combination with shotcrete.

Reinforcing steel -- Rebar that has been excessively yielded or otherwise damaged must be replaced. This is generally accomplished by removal of the damaged portions and replacing with new steel welded in place. Generally full penetration butt welding is preferred, though lap welding may be used in some cases. In any event, because of the varying heat dissipating properties of the steel which is encased in concrete and that which remains in the open, such welds will require close control of temperature. Normal procedure involves pre-heating to a temperature of approximately 200°C (400°F) prior to making the weld. Immediately upon completion, the weld area should be wrapped in asbestos to prevent rapid cooling. Also, the concrete should be removed in order to expose the rebar for a minimum of 10 to 15 cm (4 to 6 in) prior to the welding.

In some cases conventional lap joints can be made and in those cases where the reinforcing is in tension only, standard mechanical splices can be used. Where sections to be strengthened are interrupted, such as by existing columns or beams, continuity is maintained by either bypassing the steel around the interfering element or continuing the new reinforcing in holes drilled through the existing element.

Rebar dowels -- Where it is not possible to penetrate the element such as in corners or at termini, or where additional shear resistance is required, reinforcing steel dowels are secured in drilled holes. Drypack, non-shrink cementious grout and epoxy resin materials have all been used for this purpose. The epoxy resin materials have been proven most suitable [3,9,10] as they require a smaller hole, minimizing possible interference with existing reinforcing as well as being more economical. Tests have shown that epoxy-set dowels properly installed will retain their full yield capacity when embedded approximately ten times their diameter. Because increasing the embedment depth of epoxy-set dowels entails only an infinitesimal amount of additional cost, it is practical and probably advisable to do so to at least fifteen bar diameters where thickness of the existing section permits. Field proof testing of grouted bars is frequently required at a rate of from 10% to 50% of the total bars set. The frequency of such tests is often reduced, however, as the job progresses, if consistently satisfactory results are obtained. Proper performance requires that the holes be filled, preferably from the closed end outward, the bar then being pushed into the partially filled hole so that the resin material oozes out around it insuring complete contact. The bar is usually twisted slightly as it is inserted to accomplish this result. The resin material can be injected with proportioning pump in-head mixing equipment or by hand caulking guns. In either case the nozzle must be provided with a hose or tube of sufficient length to reach the bottom of the hole being filled. The installation of dowels in horizontal or overhead locations is facilitated by covering the hole with masking tape. A slit is then made in the tape through which the resin injection tube is inserted, followed by the bar, the tape acting as a barrier to prevent the material from running out. Somewhat thixotropic resin formulations are generally used for this work. Optimal hole size is the smallest that can be readily drilled and yet enable insertion of the steel. Because of the creep potential of many epoxy formulations, hole sizes more than about 13 mm (1/2 in) greater than the bar diameter should not be used.

Foundations

Structural repair and retrofitting frequently entails improvement and sometimes augmentation of the existing foundation system. Both increased dead load which nearly always results from strengthening operations, as well as potential loads resulting from high overturning forces generated in the new or strengthened shear walls during an earthquake must be considered. Where the foundation system consists of conventional spread footings or mats, the most frequent treatment involves increasing the dimension, depth or both of the existing elements. Additionally, new foundation elements are sometimes provided. This is almost always the case when new shear walls are constructed. Fig. 1 shows typical examples of foundation augmentation. Where existing depth is increased, the work usually is done in alternate segments of between 1.5 m (5 ft) and 3 m (10 ft.) in length. Conventional concrete or shotcrete is generally used in such work. Continuity is maintained by placing new reinforcing steel through the existing elements, the use of epoxy-grouted dowels, or a combination of the two.

In the case of pile foundations, additional piles may be installed or the surrounding and/or underlying soil strengthened. Because of access problems usually involved in such work, additional piles are frequently composed of a number of short sections of steel piling which are welded together. They are jacked into place using the building as a reaction, alternately jacking and welding in additional pieces. The actual pile material may be steel "H" section or steel tubing. Where steel tubing is used, dirt forced into the interior thereof is sometimes cleaned out and replaced with concrete. Cast in drilled hole concrete piling can also be provided in some cases.

Where strengthening of the soil itself is to be performed, "compaction grouting" [12, 13, 14] in the case of fine-grained soils, or chemical solidification [15, 16] in the case of relatively-permeable granular material

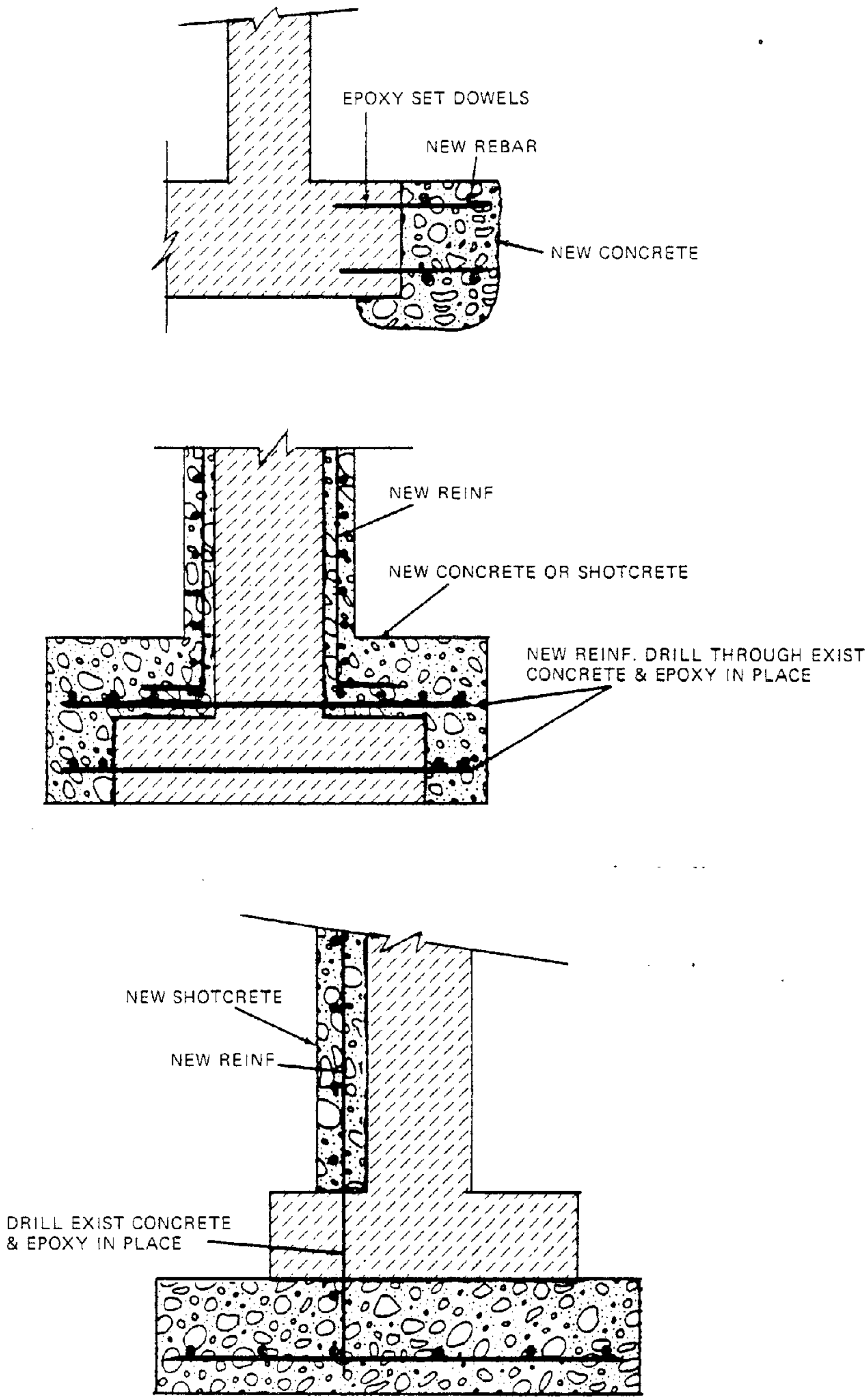


Figure 1 -- Typical foundation augmentation.

can be used. Compaction grouting results in densification of the affected soil and has been used to thereby reduce the potential for liquefaction in such soils. Reduction of liquefaction potential in granular soil by providing cohesion through chemical solidification has also been performed.

Strengthening Existing Shear Walls

Existing shear walls are frequently strengthened by the addition of section, most often utilizing shotcrete. As indicated on Figure 2, integrity of the strengthened mass is obtained by proper preparation of adjoining surface, continuation of the new reinforcing steel through the slabs, epoxy-set dowels at termini, and provision of new shear dowels at regular spacings throughout the field of the wall. Similarly, continuity is maintained at the abutments with existing walls or beams by proper preparation of the adjoining surfaces, installation of epoxy-set dowels or continuation of the reinforcing through the abutting element. Where reinforcing is continued through elements, the annular space between the rebar and hole should be filled with epoxy.

A frequent expedient involves filling existing openings in shear walls. This often requires rerouting of mechanical ducts, lines, and other components which frequently penetrate such walls. Where openings are filled in, epoxy-grouted dowels are usually installed throughout the periphery.

In the case of concrete-frame buildings with masonry infill walls, it is fairly common practice to remove one or two wythes of brick, replacing them with properly reinforced gunite. When this is done, thickened "ribs" are frequently provided around openings and at other areas where additional strength is desired. (Fig. 2) By such removal of portions of the existing masonry, it is often possible to maintain the original dimension. This also reduces additional weight imposed upon the foundation system. In such operations proper anchorage of the remaining wall components must be considered.

Because exterior facades usually are the most decorative and therefore important to preserve, such work is frequently done from the interior of the structure. Accordingly, provision must be made for proper anchorage of decorative elements. Figure 3 indicates some previously utilized methods to tie ceramic, cast stone or similar ornamentation to the strengthened structural wall section. As shown, such anchorage can be provided by the installation of bolts, wedge-type anchors, epoxy-grouted bars, and, in some cases, injection of epoxy ceramic foam. [2,4,9] Where the exterior cladding is composed of brick, stone, terra cotta or similar material, provision must be made to prevent its dislodgement during a seismic event. Epoxy ceramic foam injection has proven to be a valid method for such anchorage. [1,4,9] However, expansive cement grouts have also been used. [1,2]

Experimental work has been reported [17] wherein various precast infill panels were installed for strengthening. Wide-scale usage of such systems probably is not likely, however, due to the advanced state of development and greater economy of the other established systems. Additionally, provision of new infill panels in themselves would not provide anchorage of existing non-reinforced masonry or decorative wall cladding which, by necessity, would require either removal or some type of attachment.

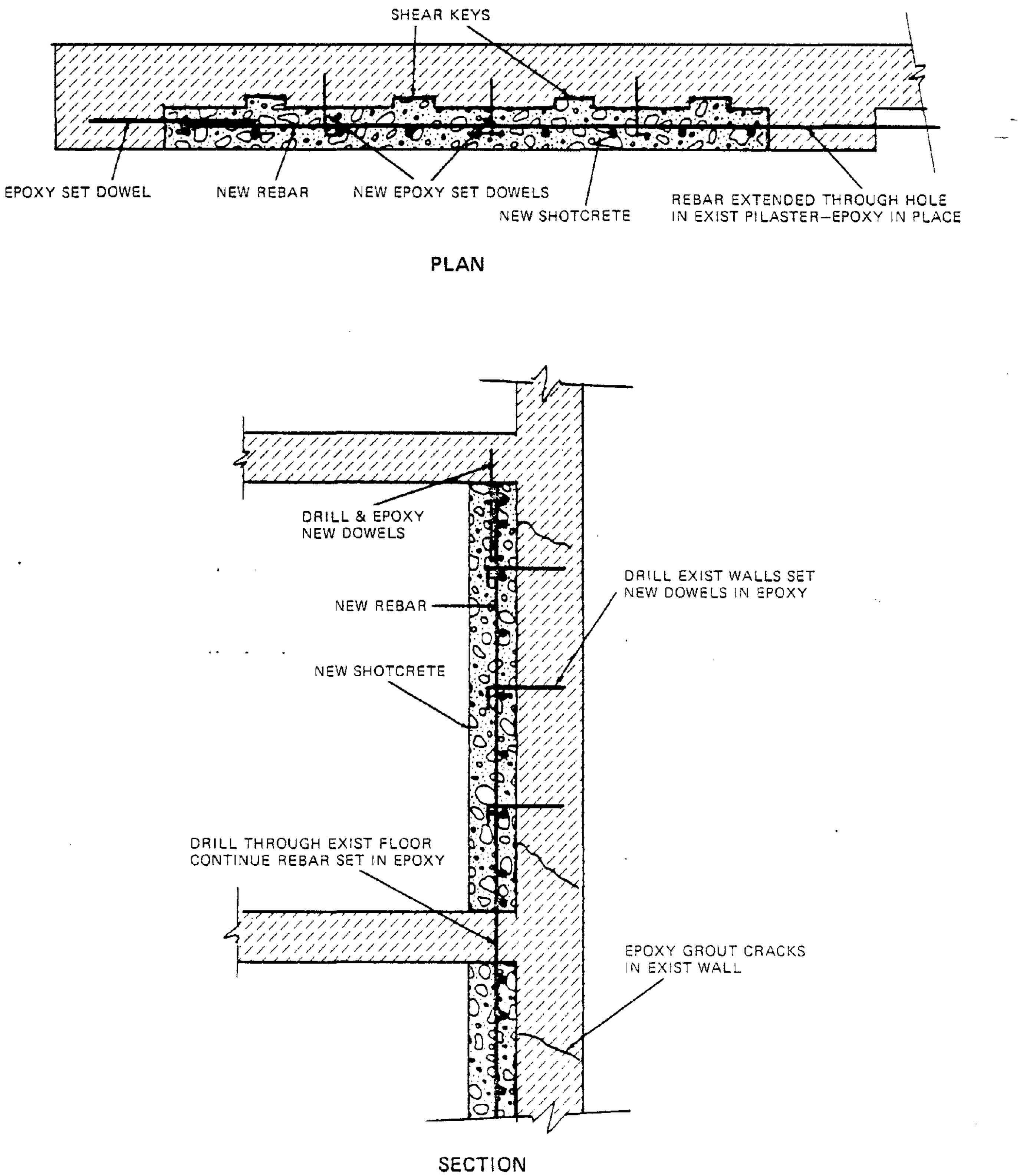


Figure 2 -- Typical shear wall strengthening.

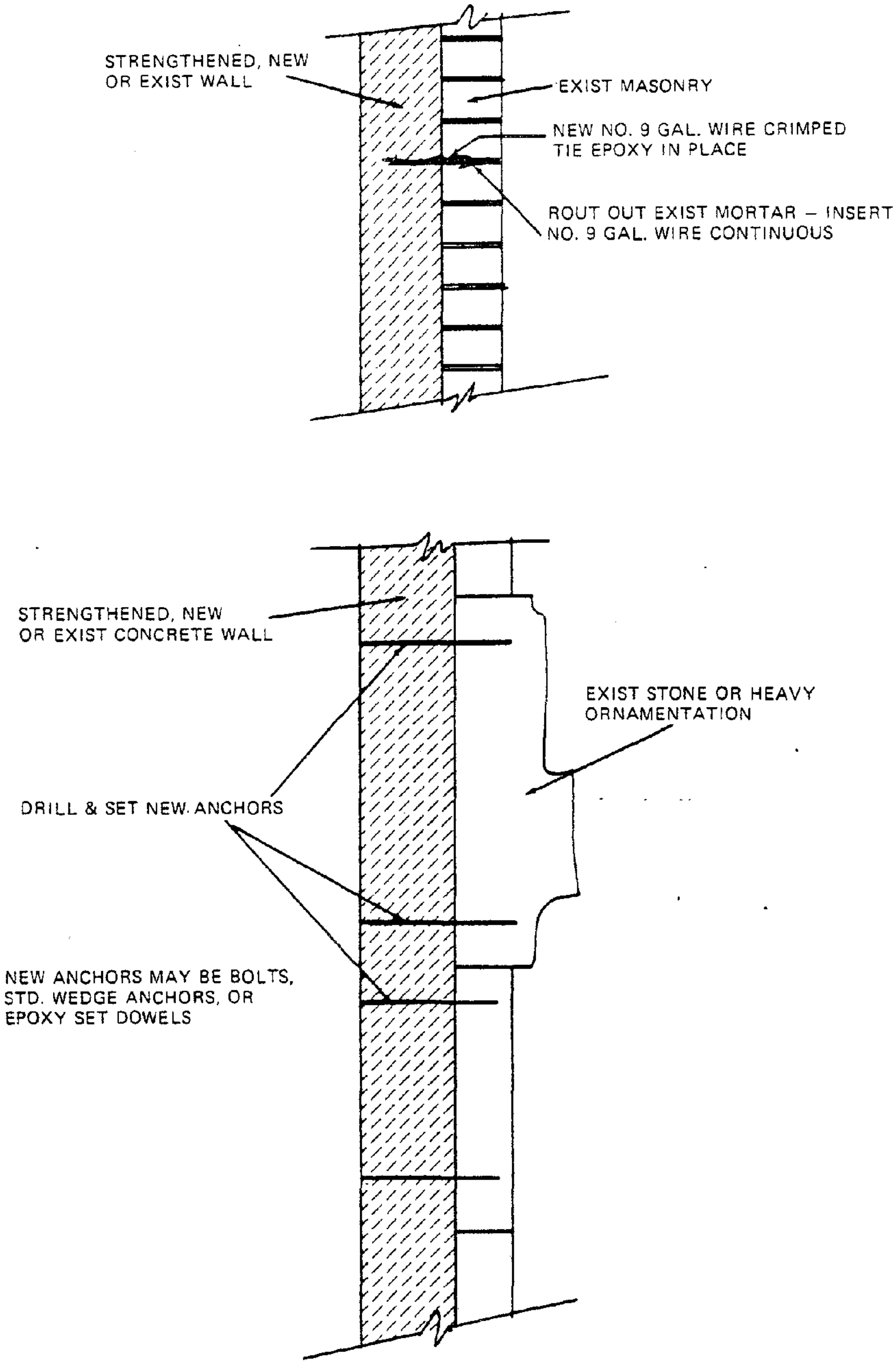


Figure 3 -- Typical methods for anchoring exterior claddings.

New Shear Walls

New shear walls are generally constructed of conventional reinforced concrete or shotcrete, although any material system which will provide the required resistance can be used. Where such elements are cast between existing concrete framing members, continuity of the reinforcing steel or the use of epoxy set dowels can be used in a manner similar to that used for strengthening existing walls. The same methods for preparing abutting surfaces are similarly utilized.

Framing Members

Existing columns and beams are frequently upgraded by addition of properly reinforced shotcrete. In order to provide a collector system to drag lateral forces to the shear walls, existing beams frequently receive special attention. Additionally, new drag members are often provided. As with the previously-discussed work, proper preparation of the surfaces to receive new shotcrete is imperative. New reinforcing steel is placed with special emphasis to insure continuity through or around other conflicting elements. Shear transfer and continuity are provided by the use of chipped shear keys, wedge anchors, or grouted bars. [2, 3, 4, 8, 9, 10, 11] Typical examples of such strengthening are shown in Figures 4 and 5.

Floor and Roof Diaphragms

Floor and roof diaphragms provide a major contribution to the distribution of forces throughout any structure. Accordingly, in strengthening applications they very frequently will require special attention. Strengthening of existing diaphragms is often accomplished by the addition of an overlay of either concrete or shotcrete. Where "change" in the elevation of the top surface cannot be tolerated, which is frequently the case, the addition of shotcrete on the underside is a frequent expedient. In some cases, stiffening ribs can be utilized. Occasionally, new diaphragms can be added by filling in abandoned shafts, stairwells, etc. The removal of existing concrete and total replacement is occasionally made as well. The preparation of surfaces and installation of reinforcing and shear resisting devices is similar to that used in the strengthening of other elements as previously discussed.

Realignment of Displaced Members

Displaced or collapsed members, assemblies or sub-assemblies can often be realigned by structural jacking. [14, 15] Unitized jacking equipment is available which permits the use of a nearly unlimited number of individual jacks operated individually or in unison from a central control console. Such equipment provides the ability to precisely realign misplaced elements without the introduction of new or deleterious stresses. Following realignment, the damaged or missing sections are replaced as previously discussed.

Anchorage of Non-Structural Elements

Fixity of parapets, cornices, sculptured figures and similar non-

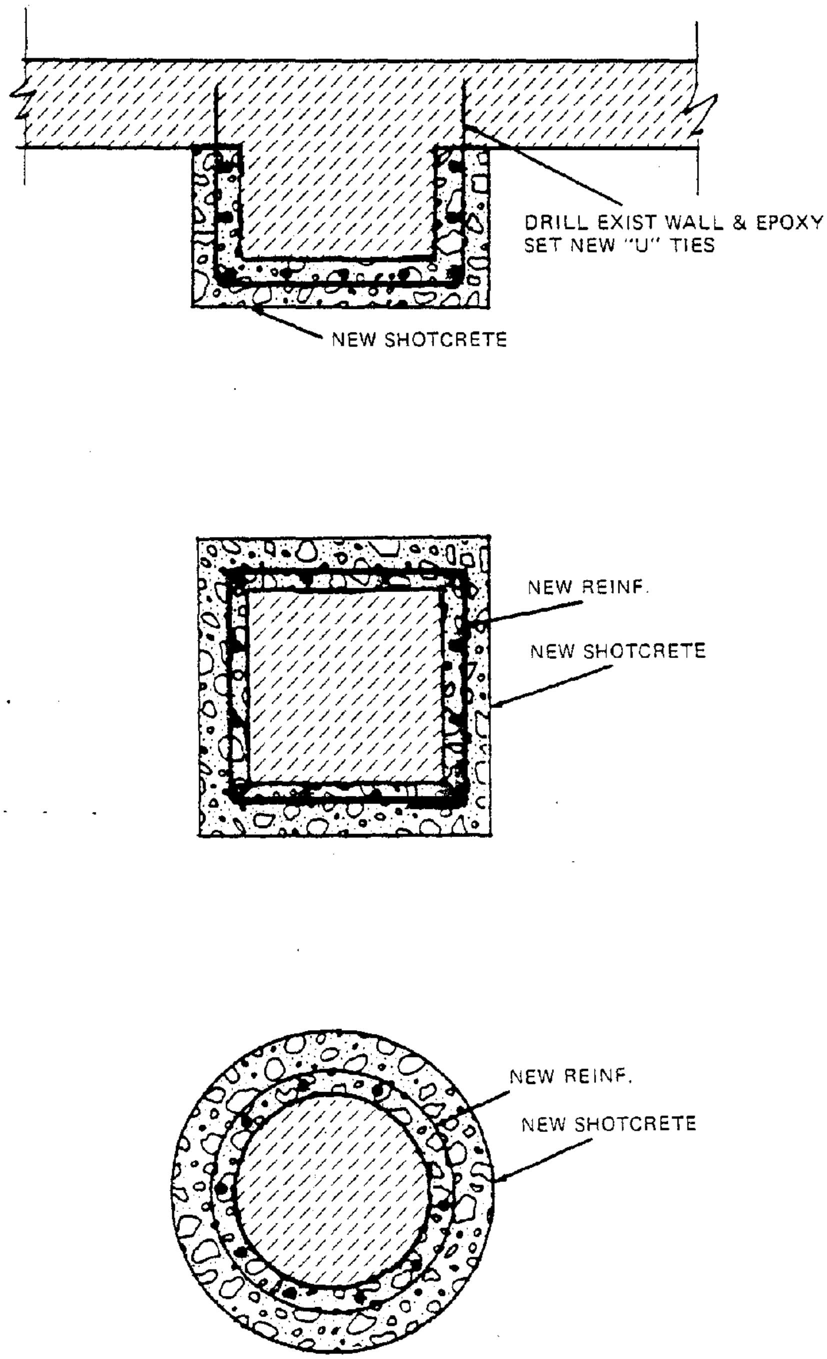


Figure 4 -- Typical methods for column strengthening.

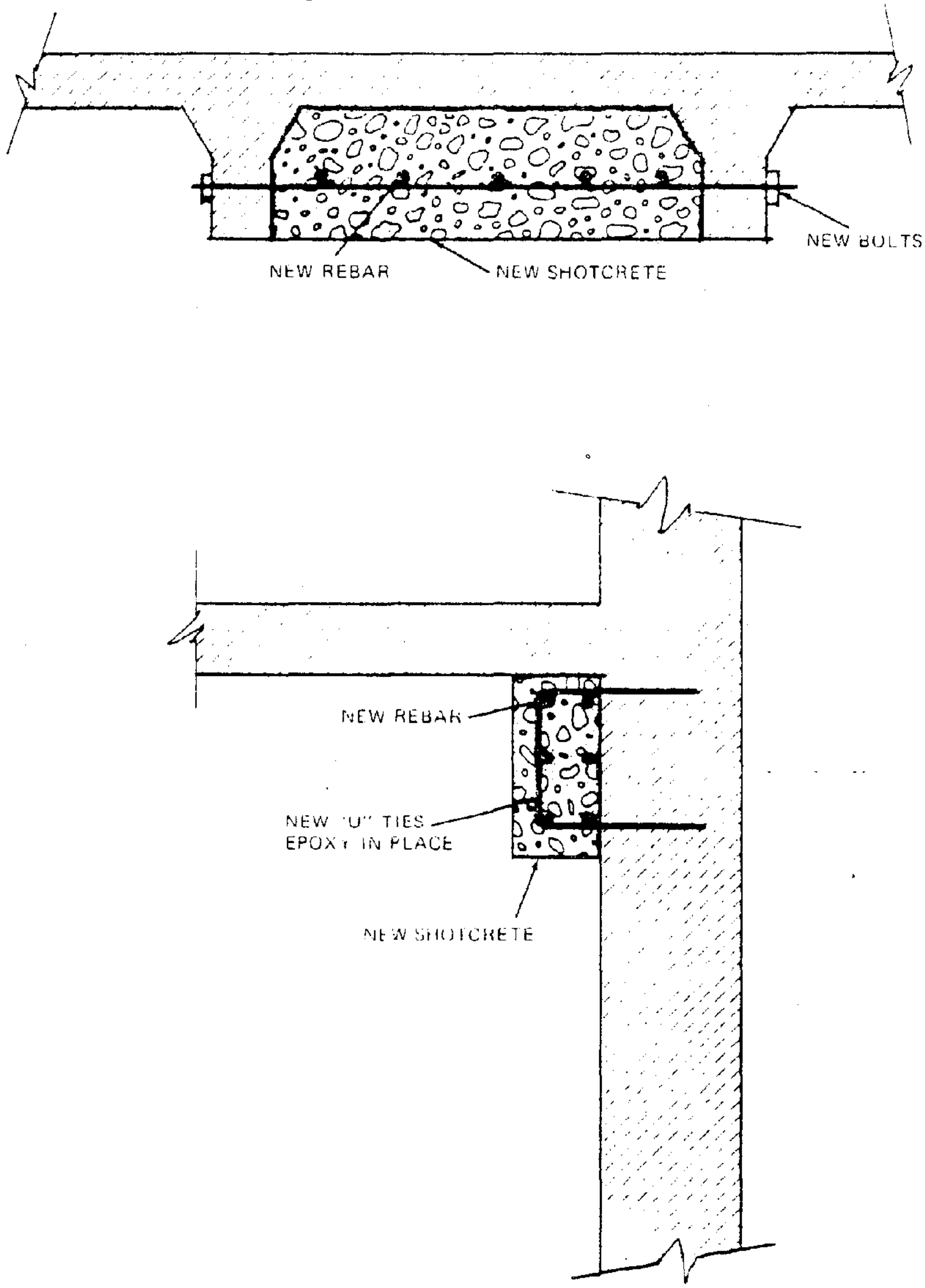


Figure 5 -- Typical methods for strengthening beams and new collector members.

structural elements is required to render a building seismically safe. Such anchorage may be accomplished by tying with wedge type or grouted anchors, bonding with epoxy mortar or similar materials, bolting or bracing with steel elements. [2, 4, 9] Figure 6 depicts typical anchorage methods. When steel is embedded within the structure it is important to assure against eventual corrosion. Hot dip galvanizing is frequently used in this regard. Additional protection is sometimes provided by encasement with concrete or epoxy ceramic foam. [9]

OBSERVATIONS AND CONCLUSIONS

Whereas historically, neither repair nor strengthening of structures has been performed on a large-scale basis except immediately following damaging earthquakes, viable methods and procedures therefor have, nonetheless, become fairly well established. In most instances, the methods have developed empirically, although in some instances laboratory or field research has preceeded actual usage. Because of existing building code requirements most of the work performed in the United States has been severely limited as to methodology. Accordingly, the materials and procedures which have become fairly well accepted, if not already covered by existing building codes, have been developed under conditions of considerable restraint, in order to obtain approved exceptions to the controlling code. In the case of large or important projects, often elaborate and costly testing programs have been performed. However, many smaller and less important undertakings, and, in some cases even large projects, have been performed using methods that all too often have been inadequate, improper, and certainly not in the best interests of the owners or the public.

Due to the infrequency of seismic events in any given area, when they do occur, design engineers, building officials and established contractors are severely limited in providing remedial treatment due to lack of experience in performing such work on a wide scale. Accordingly, formulation of a set of guide procedures for such performance is badly needed.

Epoxies and other resinous materials have received widespread acceptance in repair work in recent years. However, this field of chemistry is extremely complex and very little is understood relative to the properties or resulting behavior of such materials. Compilation of a guide, enabling identification of specific properties required for desired end results and development of appropriate analytical and acceptance criteria thereof is needed. Additionally, evaluation of the composite behavior of epoxy injected elements and assemblies, and, in particular their performance under conditions of elevated temperature (fire) or extreme exposure, is needed.

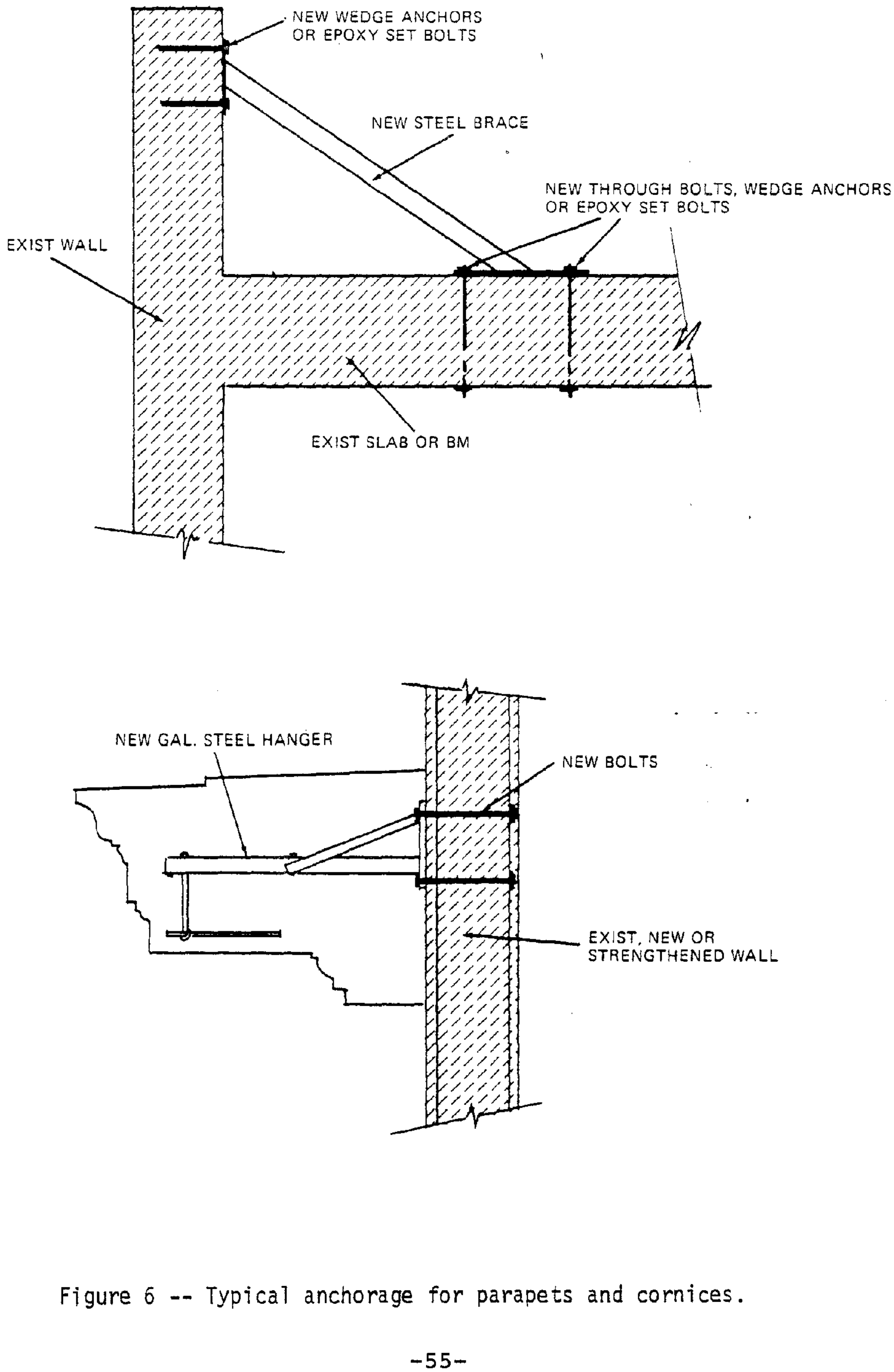


Figure 6 -- Typical anchorage for parapets and cornices.

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RETROFITTING AND REPAIRS OF EXISTING STEEL BUILDING STRUCTURES

by

Hiroyuki YAMANOUCHI*

INTRODUCTION

For the last two decades in Japan, a large number of steel building structures has been constructed not only in big cities but in local towns. It may be said that such a rapid spread of steel structures caused poor structural design and poor quality of construction which led the steel structures into much eartquake damage at the recent severe earthquakes, such as Izu-Oshima Earthquake (1978) and Miyagi-Ken-Oki Earthquake (1978). Many steel buildings in those areas required considerable amounts of repair and retrofitting for their rehabilitations. The design and construction for retrofitting of these buildings, however, would be conducted by poor knowledge and judgement due to lack of appropriate guidelines for retrofitting. Really up to that time (1978), we had not any guideline for retrofitting of steel buildings.

In June of 1978, just after the Miyagi-Ken-Oki Earthquake, a design guideline was made public by the advisory committee for Ministry of Construction [1]. This guideline was compiled to be used coupling with the evaluation method of seismic safety proposed by the same committee [2]. In the field of steel building structures, however, the experience of aseismic strengthening or retrofitting has been little, and also available test data on strengthening of damaged structures have been limited, so that the proposed guideline is limited to the description of design concepts for retrofitting.

This report describes the outline of the design guideline for retrofitting, associated with some examples of repairs which were observed after the Miyagi-Ken-Oki Earthquake.

DESIGN GUIDELINES FOR RETROFITTING

<u>General</u> This guideline covers low-and medium-rise existing steel buildings by ordinary construction method. Necessity of retrofitting will be judged from the result of evaluation on seismic safety using the criterion [2]. The result of evaluation is expressed by Eq.(1) about the longitudinal and ridge direction at each floor of a building.

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* Senior Research Engineer, Building Research Institute, Ministry of Construction, Japan

$$V_R = V \times Q \times S \qquad (1)$$

where, V_R : seismic index of structure

V : seismic sub-index of basic structural performance

Q : seismic sub-index of structural guality

S : seismic sub-index of structural profile

Here, Q-index and S-index are not related to the floor location and the direction. Q-index depends on the quality of construction and the time-dependent deterioration. S-index is represented mainly by the irregurality of floor plan profile. The value of both indexes can vary between 0.8 and 1.0. When the increase in seismic ability of a structure is recommended, Q and S-indexes can not be easily improved, because it usually costs large reconstruction. Therefore, improvement of seismic safety must usually depend upon the increase of V-index.

Contents of V-index V-index is represented by Eq.(2) for each floor and direction of a structure.

$$V = 270T \sqrt{{}_eA + {}_pA} \qquad (2)$$

where, T : natural period of building (sec.)

${}_eA$: index of elastic energy preserved in structure

${}_pA$: index of absorbed plastic energy for overall structure until i-th story collapses

Moreover, ${}_pA$ consists of the following parameters.

$${}_pA = C \cdot \alpha^2 \cdot u \cdot \phi \cdot \eta \qquad (3)$$

where, C : constant determined by number of stories of building

α : yield shear coefficient of i-th story

u : degrading factor of strength of i-th story

ϕ : coefficient of energy concentration into i-th story

η : plastic ductility preserved in i-th story

To increase the value of V-index, it is necessary to increase the values of α, ϕ, η. In the following sections, the concrete methods to do so are briefly described.

Increase of α

The yield shear coefficient α of i-story is defined by $\alpha = Qy / W$, where Qy is the yield shear strength of i-th story and W is the weight above i-th story. Therefore α of i-th story will be improved when the yield shear strength Qy is increased and/or the weight W is decreased. The latter example is shown in Fig.1.

(1) Increase of strength in rigid frames

The effective increase of story shear strength can be achieved by new columns or new frames placed within the existing structure. Reinforcements by new elements attached to existing framing elements may not be so effective because the joint action of both elements may be impeded by poor welding etc. resulted from bad conditions of construction. Such a case is shown in Fig.2.

Another method to strengthen the rigid frames is to increase of the strength of connections. As existing steel building structures be in accordance with allowable stress design, the strength of connections may not be reserved until the beam and columns reach to their full-plastic states, that is, the story shear strength of most steel buildings may be limited by the strength of their connections. Therefore, strengthening of the connections is one of effective ways to increase not only story strength but also plastic deformability of the structure

(2) Increase of strength in braced frames

In the case of braced frames, there are three types of methods to increase the strength, that is, (a) to place new bracing members within existing frames, (b) to replace existing braces with new ones, (c) to strengthen the connections of the existing braces. The method (a) may be the easiest way to realize the strengthening, however, careful attention must be paid to avoid the eccentric arrangement of braces not only in plan profile but also in section profile. Commonly concerning with the three methods, if a earthquake attacks the strengthen braced frames, increased additional stresses will be induced in the existing beams, columns and beam-to-column connections. Furthermore, similar stresses will occur, if the new braces are eccentrically set, that is, the brace lines do not pass through the beam to column intersecting points. Such stresses must be covered in retrofitting design. Fig.3 shows an example of repairs by the method (a). The method (b) is often adopted in the case of fracture of braces or brace connections due to an earthquake. In this case, sectional areas of new braces may be usually larger than those of old braces, so that attention must be paid to detailing of brace connections not to fracture in the duration of earthquake motions.

The mothod (c) will be effective as well as in case of rigid frames. Throught the field investigation of the damage of braced steel frames due to the recent earthquakes, it became clear that structural design and/or workmanshop about the jointed parts of braces were inadequate. The main factors which influence on the strength of jointed parts of braces are; 1) the strength of the effective sectional area of the braces at bolted parts, 2) the strength of fasteners, and 3) the strength of gusset plates including their welded parts. As for the strengthening of brace joints the Ministry of Construction recommended (1979) that the ultimate strength of the jointed parts of braces must be 1.2 times larger than the general yield strength of braces at the design of new buildings. It is desirable to check up whether existing steel buildings be in accord with this rule or not.

Increase of η In order to increase the ductility of a certain story, the framing members must be improved so as to exhibit enough plastic deformations without degrading of load carrying capacities. For this purpose, obviously, local buckling of plate elements and lateral buckiling of beams, which may prevail in the early stage of plastic range should be prevented. In case of local buckling the sections subject to large compressive stresses should be braced by suitable stiffeners, ribs, or cover plates. Lateral buckling should also be prevented by additional lateral supports to shorten the support intervals.

The strengthening of connections included in beams and columns also can promise the increase of ductility. For this aim, the ultimate moment capacity of connections must be improved to exceed the general yield moment capacity of the connected members.

Increase of ϕ The factor ϕ is directly related to the distribution of yield shear coefficient α along the height of a building. If the profile of the strength distribution along the height is different from a optimum pattern, that can be specified by response analyses, the input energy due to an earthquake will concentrate in the relatively weaker stories. This concentration gives relatively smaller values of ϕ to those stories. In order to increase the value of ϕ , the pattern of the strength distribution must be put close to the optimum distribution by the previously mentioned methods about α . Fig.4 shows an example of repairs concerning with the parameter ϕ

CONCLUDING REMARKS

As it is two short years since the publication of the guideline for aseismic retrofitting, the practical applications to existing steel buildings are very few in accordance with the guideline. Meanwhile, judging from the field investigations on the damage due to the recent earthquakes, it can be said that a considerale number of steel buildings may need strengthening. But unfortunately, information on practical retrofitting or repairs, at present, has not be collected systematically. Moreover, researches on retrofitting of steel structures have not yet interested engineers and not begun, in Japan. Thus, the amount of information and experience on steel structures is considerably less than that on reinforced concrete [3]. The most significant reason for this situation may be that public or official steel buildings are very few, so that the avilable and reliable data, or requests of experiments for retrofitting can not be obtained. This being the case, it is expected that the guideline may raise a subject on retrofitting for steel buildings in Japan.

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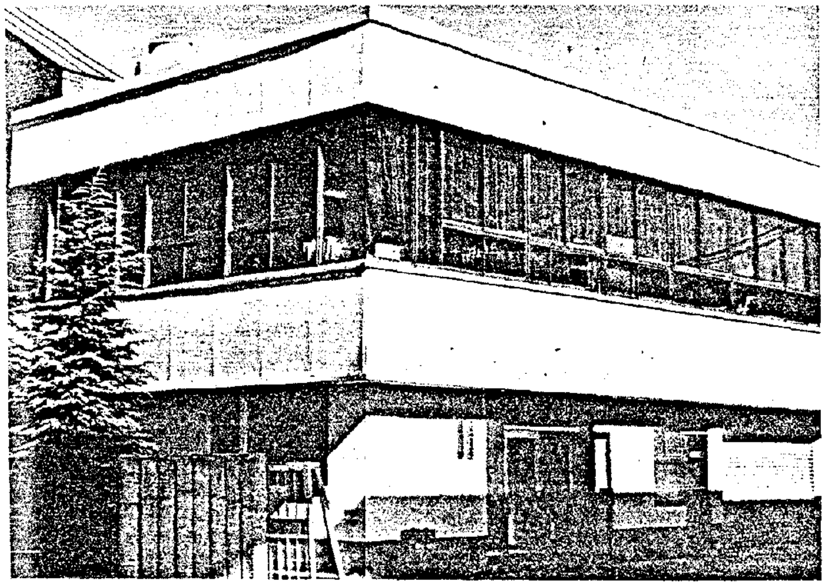

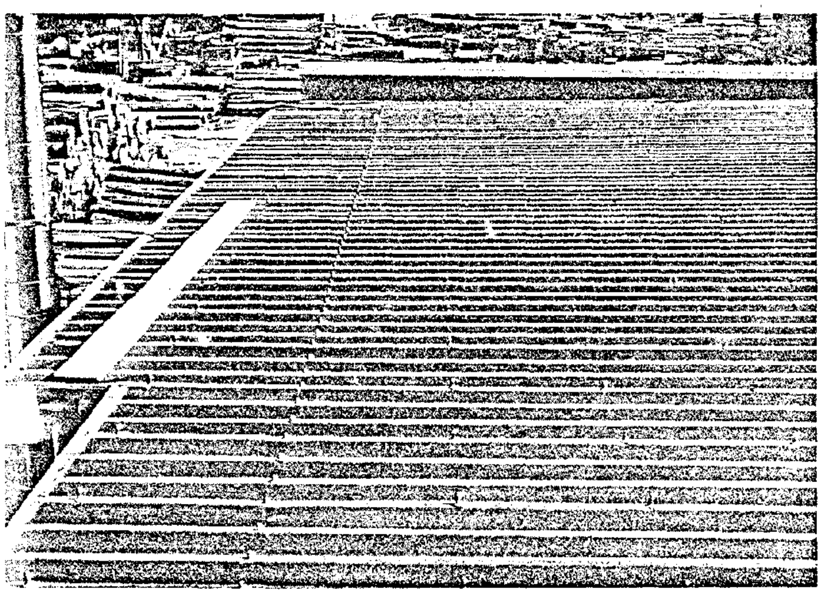

Fig.1 Repair for Improvement of Yield Shear Coefficient

Before the Miyagi-Ken-Oki Earthquake, this office building had a concrete flat slab as the roof. Because the braces were damaged due to the earthquake, the roof was replaced with light steel slates.

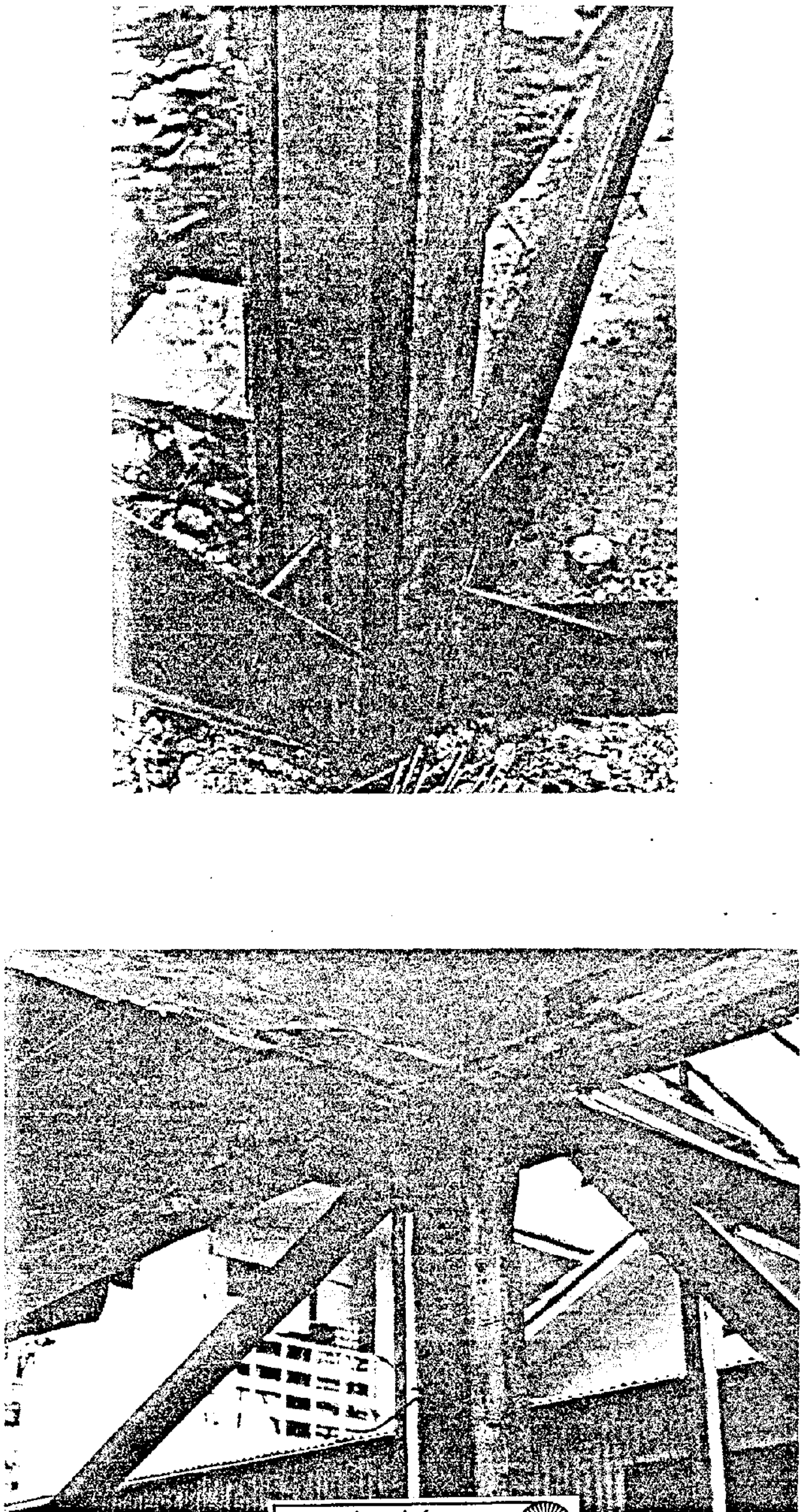

Fig.2 Strengthening of Columns by New Slender Columns

New columns were attached by intermittent welding, so that the joint action can not be expected.

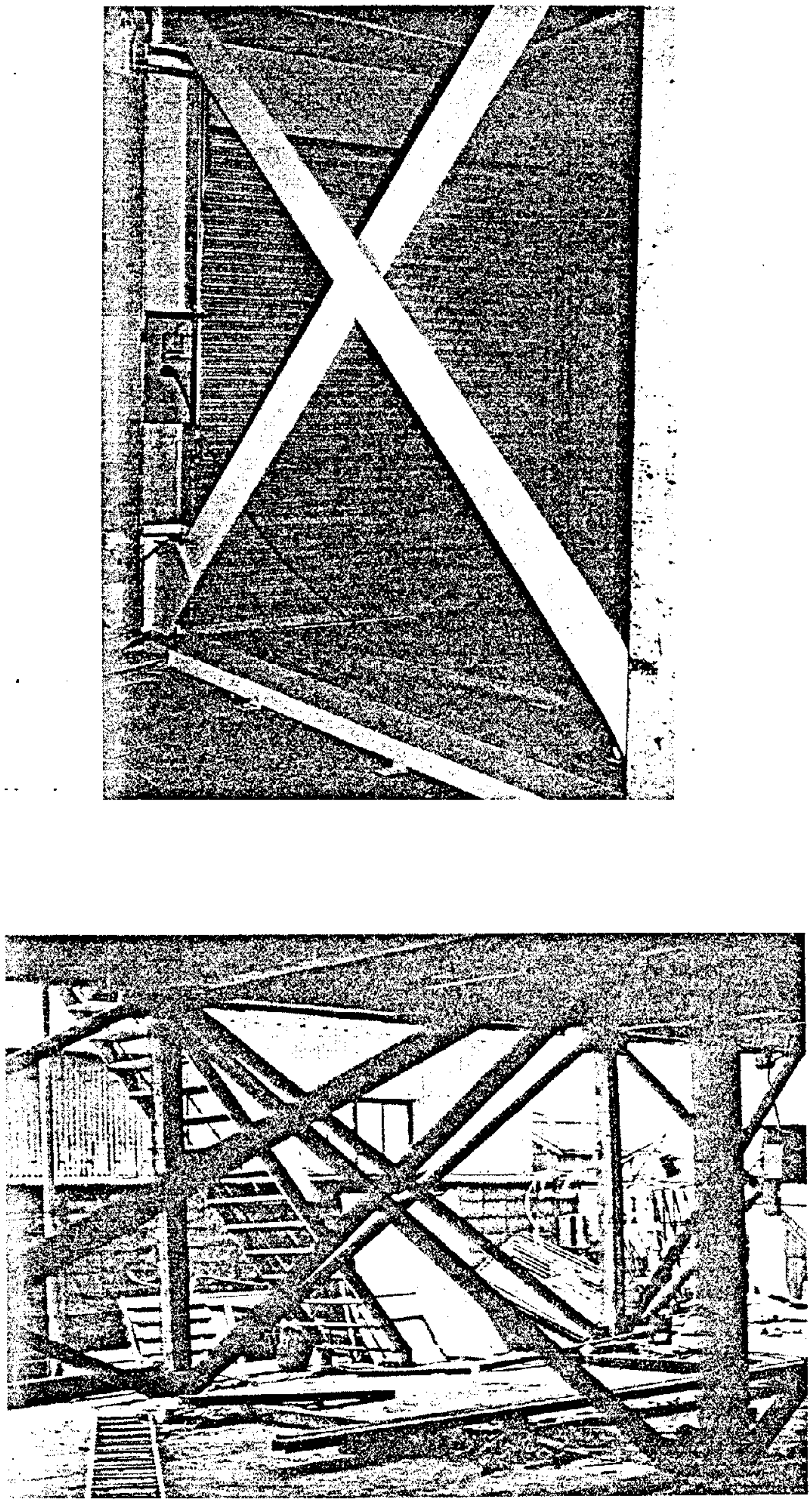

Fig.3 Placement of New Braces

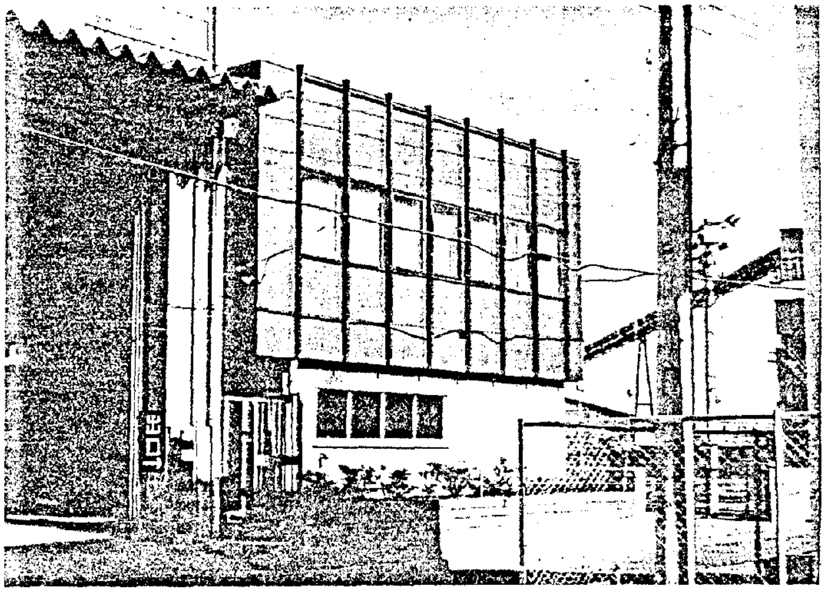

Fig.4 Conservative Reconstruction

This office building was four storied before the Miyagi-Ken-Oki Earthquake. The damage concentrated in the third story due to the discontinuous distribution of yield shear coefficient α. The story drift reached to about 1/20 of the story height. The structure above the third floor level was removed by the conservative consideration for aseismic safety.

SEISMIC EVALUATION AND REHABILITATION OF HUD RESIDENTIAL BUILDINGS

G. Robert Fuller, P.E., M-ASCE
Structural Engineer
Office of Architecture and Engineering Standards
Office of Housing
U.S. Department of Housing and Urban Development

ABSTRACT

On June 23, 1976, HUD awarded a contract to S.B. Barnes and Associates to develop a "Methodology for Seismic Evaluation of Existing Multistory Residential Buildings." Included in the methodology were repair, retrofit and strengthening techniques to be used in rehabilitation of projects under HUD programs. A three-volume manual was published in November 1978. This paper is a report of several HUD projects evaluated by private sector consulting engineers using the cited methodology.

BIOGRAPHY

G. Robert Fuller, Structural Engineer in HUD's Office of Housing since 1970, has principally been involved in the review and acceptance of multistory concrete and masonry residential buildings. He has also been instrumental in developing design criteria for analyzing the general structural integrity of masonry and precast concrete bearing wall buildings, including resistance to high wind and earthquake forces. Mr. Fuller is a member of American Concrete Inst., Prestressed Concrete Inst., Post-Tensioning Inst., American Society of Civil Engineers, Federal Interagency Committee on Seismic Safety in Construction, Building Seismic Safety Council, and Council on Tall Buildings and Urban Habitat.

Key Words: Earthquake Resistance; Rehabilitation; Renovation; Residential; Retrofit; Seismic Evaluation; Strengthening Techniques.

INTRODUCTION

HUD awarded a contract on June 23, 1976 to S.B. Barnes and Associates of Los Angeles, Calif. to develop a "Methodology for Seismic Evaluation of Existing Multistory Residential Buildings." Principal developers of the Methodology were Clarkson W. Pinkham of S.B. Barnes and Assoc., and Gary C. Hart of J.H. Wiggins Co., Redondo Beach, Calif. An initial report of this project was made by the author at the Tenth Joint Panel Meeting, May 1978.

A manual was published in November 1978, in three volumes:

Vol. No. 1 - Methodology,

Vol. No. 2 - Computer Users' Manual,

Vol. No. 3 - Examples.

Included in the "Methodology" and "Examples" portions of the manual are repair, retrofit and strengthening techniques which can be used in rehabilitation of projects under HUD programs. Several HUD projects evaluated by private sector consulting engineers using this methodology are reported herein.

GENERAL

The response of an existing building to earthquake motions reflects the performance level inherent in the codes, standards, and construction practices in existence at the time of the design and construction of the building. Building practices continually improve during the life of a building, reflecting the advancement of the state of knowledge. Thus, the implied margins of safety changes, depending on whether a comparison is made with the code in force at the time of design or with the current code. Deterioration and improper alterations during the service life of the building affect the actual margin of safety provided by the building. The need exists, therefore, to evaluate the potential seismic hazard of each building proposed for rehabilitation. Following such an evaluation, the cost of appropriate strengthening or retrofit procedures has to be calculated so that the feasibility of various schemes to mitigate unacceptable hazards can be determined.

The manual describes the survey and evaluation procedures necessary to determine the seismic hazard of existing multistory residential buildings. The method of evaluation is given in terms of the behavior of the critical structural elements in the building. The determination of this behavior requires an analysis of the structural response of the building to prescribed forces and the determination of the strength of the critical members and connections of the earthquake resisting system of the building. The general methodology is applicable to any complete set of earthquake resisting design standards and to most types of building systems.

Evaluation is based on the 1973 Edition of the Uniform Building Code, UBC 73, with some modifications as defined herein, specifically the earthquake forces specified in Chapter 23 and Chapters 24, 25, 26, 27, and 28 stipulating the design criteria for masonry, timber, concrete, steel, and aluminum respectively.

The methodology is limited to the evaluation of the following multistory residential building types: ("Masonry B" is masonry construction conforming to Sections 2414, 2415, 2418 of UBC 73. "Masonry A" is all other types of concrete or brick masonry.)

1. Non-prestressed concrete frame with beam and slab floor or concrete flat slab floor with:

   a. Infill non-bearing walls of Masonry A.
   b. Bearing walls of Masonry A.
   c. Infill non-bearing walls of Masonry B and/or concrete.
   d. Bearing walls of Masonry B and/or concrete.

2. Masonry A bearing walls with light wood floor and interior wall framing.

3. Masonry B bearing walls with light wood floor and interior wall framing.

4. Steel moment resisting frames with floors composed of non-prestressed concrete slabs or of steel decks with reinforced concrete fill and either:

   a. With no additional seismic resistance, or
   b. In combination with:

      (1) Masonry A walls,
      (2) Masonry B walls,
      (3) Concrete walls, or
      (4) Braced bays.

5. Steel frames (vertical load) with floors composed of non-prestressed concrete slabs or of steel decks with reinforced concrete fill and either:

   a. Masonry A walls,
   b. Masonry B walls,
   c. Concrete walls, or
   d. Braced bays.

6. Conventional wood frame (up to 4 stories).

The earthquake effects on a structure that are evaluated using the methodology are those resulting from shaking. Not evaluated are the earthquake effects which produce foundation settlements or soil failures, ground lurching, liquefaction, surface trace of earth faulting, failure of the slope beneath the structure, tsunamis, seiches or inundation resulting from the failure of dams or reservoirs.

The following step-by-step procedure from the Manual was used to evaluate the seismic resistance of the structures:

1. Obtain all basic data and complete Data Collection Forms.

2. Decide, without further analysis, whether or not strengthening is feasible.

3. Extract criteria from UBC 73 and generate necessary basic input data for analysis.

4. Distribute basic loads from UBC 73 criteria to the structure.

5. Generate appropriate structural model.

6. Determine load effects on critical elements of the structure (axial, flexural, shear and torsional stress on components and connections).

7. Determine resistance capacity of critical components and connections using basic data and code criteria.

8. Determine critical stress ratios.

9. Determine whether strengthening is necessary.

10. Identify strengthening procedures and details.

11. Determine costs of strengthening procedures.

12. Decide on whether or not to proceed with strengthening or to provide further evaluation iteration.

METHODS OF STRENGTHENING AND REPAIR

General Considerations

Shear Walls. Increased shear wall capacity may be accomplished by adding new walls, adding reinforcement or increasing the thickness. The effect of added weight and stiffness on the foundation must be considered. Added mass and stiffness also will affect the building response to ground motion by increased equivalent forces and by altering the eccentricity between centers of mass and rigidity. Use of bracing may reduce the problem of added weight.

Structural Steel Frames. Columns and beams may be strengthened by adding plates. Column splices and truss joints are frequently critical and usually can be strengthened by modification.

Concrete Frames. Individual members can be encased with pneumatically placed concrete (shotcrete) with added reinforcement. Top bars can be installed in beams by cutting into the floor or roof slabs with added ties around the beam encased with shotcrete. Strengthening by post-tensioning can sometimes be done when access is available for anchorage. A combination of post-tensioning and shotcrete can also be used.

Infill or Filler Walls. If badly cracked, brittle filler walls should be replaced or reinforced to act as shear walls. The effect of their stiffnesses should be considered in the design of lateral force resisting system. Walls can be isolated on three sides so they will not affect the lateral forces resisting system.

Existing Stresses. The stresses in members prior to strengthening should not be ignored when evaluating the methods to be used. However, problems may be minimized by shoring.

Removal of Upper Stories. Where a multistory building is found to be hazardous, it may be feasible to remove one or more of the upper stories. The removal of upper stories reduces the building mass but also shortens its natural period of vibration. This method may be feasible when strengthening the lower stories is not possible.

Effects on Stiffness from Strengthening. When selecting the method of strengthening, consideration should be given to effects of concurrent stiffening of the building. Stiffening the building may require higher levels of resistance than indicated by the analysis of the initial building.

Joinery. Details of joints between new and existing elements should be chosen to provide for adequate transfer of all forces between elements.

Reinforced Concrete Frames and Walls

The most frequently found damage in reinforced concrete buildings is cracking or crushing. Where the extent of damage is great, consideration should be given to replacement in total or in part. This usually requires temporary shoring. However, there are several successful methods of repair, discussed in the Manual, which should be considered:

1. Epoxy injection.
2. Epoxy mortar.
3. Foam epoxy.
4. Cement grout and mortar.
5. Anchorage of Reinforcing Steel.
6. Repair of Reinforcement.
7. Wall anchorage.

Also presented are strengthening methods for reinforced concrete. Replacement of concrete members with stronger elements sometimes may be done without materially affecting the stiffness of the lateral force-resisting system. However, other methods have also been successfully employed (See Figs.1, 2 & 4):

1. Pneumatically Applied Concrete (Shotcrete).
2. Post-Tensioning.
3. Added Concrete.
4. Infill Walls.
5. Bracing.

Masonry Structures

Repair and strengthening of masonry bearing wall buildings is usually similar in many ways to that described for concrete structures (See Figs. 1, 3 & 4). However, concrete masonry units have large voids that may be utilized in the strengthening method.

Some successful methods of repair and strengthening have involved the use of:

1. Shotcreting.
2. Vertical prestressing.
3. Reinforced portland cement stucco or plaster.
4. Structural steel bracing.

Structural Steel Frames

Repair techniques used successfully for severely distorted members have been flame or mechanical straightening. Replacement of the damaged portions also has been employed. Defective bolts and rivets can also be removed and replaced.

Strengthening methods include:

1. Reducing the unbraced length of long columns by use of intermediate bracing.
2. Adding plate elements to beams and columns.
3. Reworking or replacing connections.
4. Adding knee braces.
5. Adding new braced frames.
6. Adding anchor bolts or weld plates to connections.

Wood-Framed Buildings

Usually it is more economical to replace wood members than to repair them. However, some defects can be repaired by using epoxy injection or by splicing on supplemental members.

Replacement of members by stronger members is frequently an economical method of strengthening. Members may also be spliced onto existing members to increase the strength. Other strengthening methods that should be considered are:

1. Knee braces.
2. Use of steel posts and tension rods to form a king or queen post truss with a solid timber beam.
3. New anchors or connections between floor and roof elements, and vertical load-carrying elements.

Horizontal Diaphragms

Recommendations are contained in the Manual to develop diaphragm action in concrete slabs, wood floors and roofs, and steel deck systems (See Figs. 5 & 6). Both repair techniques and strengthening methods are presented in the following general areas:

1. <u>Concrete Slabs</u>:

   a. Repairs:

      1) Replacement
      2) Epoxy injection
      3) Other (See "Reinforced Concrete Frames and Walls")

   b. Strengthening:

      1) Dowels, connections, and anchorage.
      2) Added shear walls to reduce spans.
      3) Added concrete.
      4) Other (See "Reinforced Concrete Frames & Walls").

2. <u>Wood Floors and Roofs</u>:

   a. Repairs:

      1) Replacement of sheathing
      2) Additional anchors
      3) Additional connections

   b. Strengthening:

      1) Plywood blocking
      2) Renailing
      3) Additional plywood or diagonal sheathing overlay
      4) Horizontal steel bracing
      5) Anchorage of diaphragms to walls with steel plate ties and bolting.

3. <u>Steel Deck</u>:

   a. Strengthening:

      1) Modify connections to supports.
      2) Replacement of deck.
      3) Horizontal steel bracing.
      4) Additional connections between floor diaphragm and vertical load-carrying elements.

<u>Foundations</u>

Overturning effects from earthquake forces create positive and negative pressures on footings. The added compressive force may create overloads which can produce settlement by consolidation of the supporting soils. Where a building is founded on loose or not very dense soils, the shaking of the ground in itself may consolidate the soil, producing settlement of the supported structure. Liquefaction may occur during earthquakes in some types of soils containing excessive moisture.

These soil conditions may cause failure of the foundation systems or distress in the superstructure. Therefore a thorough evaluation of soil characteristics and an analysis of the soil/structure interaction may be required. After an earthquake, cracks in concrete or masonry foundation walls and footings may need to be repaired. Similar techniques to those described for repair of concrete and masonry elements may be employed.

Remedial action or strengthening methods are described in the Manual. Only two methods are described - underpinning and soil stabilization (See Fig. 7).

1. Underpinning: Where sizes are inadequate, footings may be increased by underpinning or they may be removed and replaced. New footings (caissons or piles) may also be installed on each side of an inadequate existing footing, with beams installed to carry the load. The portion of the building which has settled should then be jacked into position.

   For strengthening of pile footings that have settled, it may be necessary to install additional piles and a new pile cap. This involves temporarily supporting the existing loadbearing element and then jacking it into position.

   Consideration should also be given to a combination of additional piles and soil stabilization. This may reduce the number of piles required.

2. Soil Stabilization: There are many methods of soil stabilization and compaction that are used, such as pressure grouting or intrusion grouting with cement grout or chemicals. However, a thorough analysis of the soil is usually required, since some stabilization techniques are not effective for certain soils.

   Pressure grouting, in some cases, may be used to raise or level footings or floor slabs. When soil-cement grout is used, the method is also called "mudjacking."

   There are some instances in which chemical grouting has been successfully used in dry, granular, or fractured soils. This method should be used only when the soil chemistry is appropriate.

Non-Structural Elements

Earthquake forces applicable to most non-structural elements (elements not a part of the lateral force resisting system) are listed in UBC 73, Tables Nos. 23-C and 23-J. Elevators and mechanical equipment are not explicitly covered in UBC 73 and other codes should be consulted as a guide. Forces specified in the tables are for new construction. When deciding whether or not to strengthen existing non-structural elements, their location and their hazard to life safety during an earthquake should be considered. Guidelines for the repair and strengthening of some of the non-structural elements frequently found in residential buildings are given in the Manual.

The following non-structural elements are described:

1. Parapet Walls (See Fig. 8).
2. Masonry Veneers.
3. Appendages.

4. Non-Bearing Partitions (Fig. 9)
5. Ceilings and Light Fixtures.
6. Fire Escapes.
7. Essential Equipment.

COST ANALYSIS OF REMEDIAL MEASURES

Purpose

The purposes of the cost analysis of seismic rehabilitation, as contained in the Methodology, are:

- Combined with financing costs, other required upgrading costs, the costs of temporarily vacating the building, etc., the seismic rehabilitation cost analysis can be used to determine the economic feasibility of the project as a whole.
- To assist in the determination of the most economical engineering solution by comparing several alternative solutions.
- To determine a level of rehabilitation within a given budget when 100% code upgrading is not economically feasible. (25%, 50%, 75% code compliance).
- To set up a budget for future design and construction.

Preliminary Cost Analysis

After the Data Collection stage, it may be obvious that major strengthening will be required to bring the structure up to reasonable compliance with UBC 73. A preliminary, rough, structural analysis may show that the major critical elements are inadequate. From a rough estimate, the decision can then be made whether or not to proceed with further structural analysis and a more detailed cost estimate.

Detailed Seismic Cost Analysis

The detailed cost analysis for seismic structural rehabilitation should only include work which falls into one of the following categories:

1. The work item being considered contributes <u>directly</u> to the seismic structural strengthening,
2. The work is required in order to install items defined in (1),
3. The work is required in order to return the building to its original condition.

These categories of work to be considered in a seismic rehabilitation project are specifically delineated in the Manual.

HUD REHABILITATION PROJECTS

Analysis Procedure

Volume III of the Methodology contains two detailed analyses of actual buildings. Both computer and hand calculations were conducted to determine the resistance capacity of the buildings as they presently exist.

First, the period of the building is computed; the base shear is determined; and loads are calculated in conformance with UBC 73. Critical elements are then analyzed and unit stresses are computed.

Next determined is the best possible method of rehabilitation, ~~and~~ followed by a preliminary analysis and cost calculation.

Finally, critical stress ratios and cost figures for 100% compliance with UBC 73 are determined. Cost figures for 75%, 50%, and 25% are also tabulated.

Building Number One

This building is a six-story apartment building with basement. The building is 39' (12m) x 112' (34m) in plan and 63' (19m) high. It has unreinforced brick masonry exterior walls with two interior wood framed bearing walls and a flexible, diagonally sheathed wood diaphragm floor system. Basement walls are reinforced concrete.

A 'K' factor of 1.33 was used to determine the base shear. After an examination of the masonry, an allowable shear stress of 7.5 psi (51.7 kPA) was chosen as being appropriate. The building is rectangular in plan, with diaphragms having a span-depth ratio of about three (L/d = 3).

Elevations of exterior walls were drawn, indicating that first story strength and stiffness would be a problem. Critical elements were chosen for specific analysis. A wall analysis similar to those used for new buildings was made. The members of the building frame were modeled and the elastic distribution of the shear on each wall was determined. Shears and moments resulting from this analysis were calculated.

Various potential strengthening methods were considered, and the following were chosen for a detailed cost analysis:

1. Diaphragm span reduction by providing interior transverse walls or frames as vertical load resisting elements.
2. Reinforcement of exterior walls by removing one wythe of brick and replacing with reinforced shotcrete.
3. Interior reinforced concrete or reinforced shotcrete lateral load-resistant, transverse shear walls.
4. Transverse moment-resistant structural steel frames or X-braced steel frames for interior lateral load resistance.

The decision for this building was to strengthen the exterior walls with shotcrete and to provide steel moment-resistant bents in the transverse direction on one side of the building corridor. The lateral load-resistant frames were located to minimize room layout problems.

The first story flexibility and weakness problem was corrected by filling in window openings with reinforced shotcrete, consistent with room layouts.

Critical stress ratios, strengthening methods and cost data for 100% compliance with UBC 73, as well as for 75%, 50%, and 25% were tabulated. The summary of the cost of structural modification was provided to assist in decision making.

Very little difference in cost was found in providing for 25% or 100% of UBC 73 forces, This indicates that in some cases design forces have little effect on the cost of providing earthquake resistance.

Building Number Two

This building, analyzed by using the Methodology, is a six-story and basement hotel built in 1926. It is L-shaped above the second floor. The structural frame consists of reinforced concrete beam, slab and girder construction. South and east walls are reinforced concrete filler walls; the two street front walls have filler walls of unreinforced brick; and the basement walls are of reinforced concrete.

The base of the building is 110' (34 m) x 200' (61 m) in plan and 81' (25 m) high. A 'K' factor of 1.0 was used to determine the base shear, and rigid diaphragm floors were assumed. The masonry was considered somewhat better than in Building One, so an allowable shear of 10 psi (69 kPa) was used.

The structure is complicated in many ways. The west edge of the building is skewed with respect to the other sides. Upper floors in the "L" shaped tower provide an eccentricity of mass to the base structure. Elevations also indicated that the first story flexibility would require analysis. However, since the south and east walls are solid, torsional forces would be induced.

A force diagram on the building layout indicated the severity of this first floor eccentricity. Distribution of the base shear and story shears were calculated and tabulated. A compatible design level for diaphragms was also determined.

This building, having rigid diaphragms and plan eccentricities, required a torsional analysis to determine appropriate design shears on each wall. Once the forces on each wall were determined, a wall analysis was performed. Critical stress ratios were tabulated to assist in determining the method of strengthening.

For strengthening, interior shear walls were suggested. Exterior masonry walls were proposed to be strengthened with 6" of reinforced shotcrete in place of one wythe of brick. After designing the appropriate strengthening methods, an analysis was again performed. The final tabulated critical element stress ratios indicated that further strengthening was not required.

As part of the required strengthening, such detail items as struts to shear walls and reinforced diaphragm chords were included. First story archways were partially filled to enable continued functioning of first-story shops.

The final cost analysis was tabulated to provide the data necessary for final decision making. Again, strengthening methods and related cost data were tabulated for 100%, 75%, 50%, and 25% compliance with UBC 73, Seismic Zone 3.

CONCLUSION

The "Methodology for Seismic Evaluation of Existing Multistory Residential Buidlings" has proved to be an effective tool in analyzing existing buildings proposed for rehabilitation under HUD programs. Several buildings in the San Francisco and Los Angeles areas have been analyzed and strengthened using procedures outlined therein.

Tabulated cost data related to percent compliance with the 1973 Uniform Building Code has also proven to be invaluable to program administrators and building officials charged with the responsibility of deciding whether to rehabilitate or destroy a building. This data is often used to decide how much strengthening is required and how much is economically feasible. It can also be a major factor in determining acceptable risk.

* * * * * * * * * *

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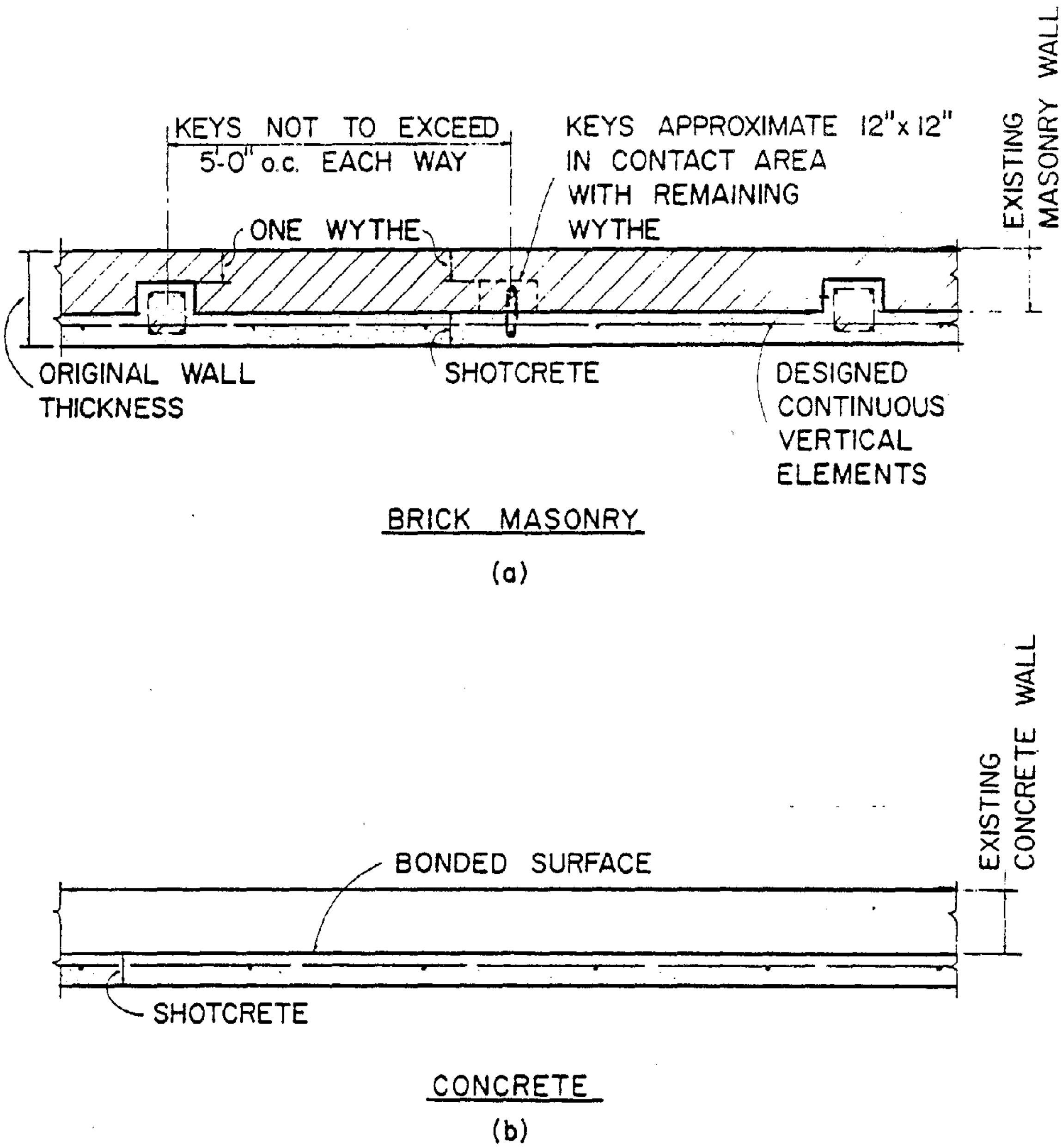
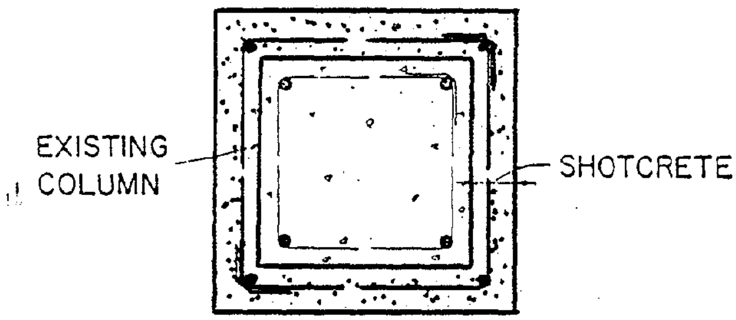
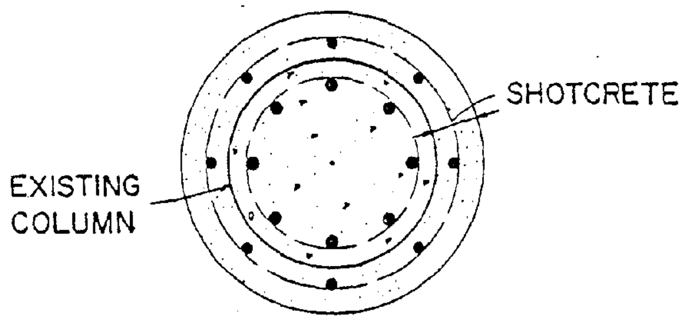


TYPICAL WALL STRENGTHENING

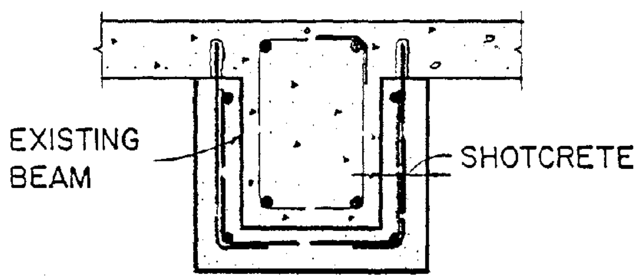
FIGURE NO. 1



(a)



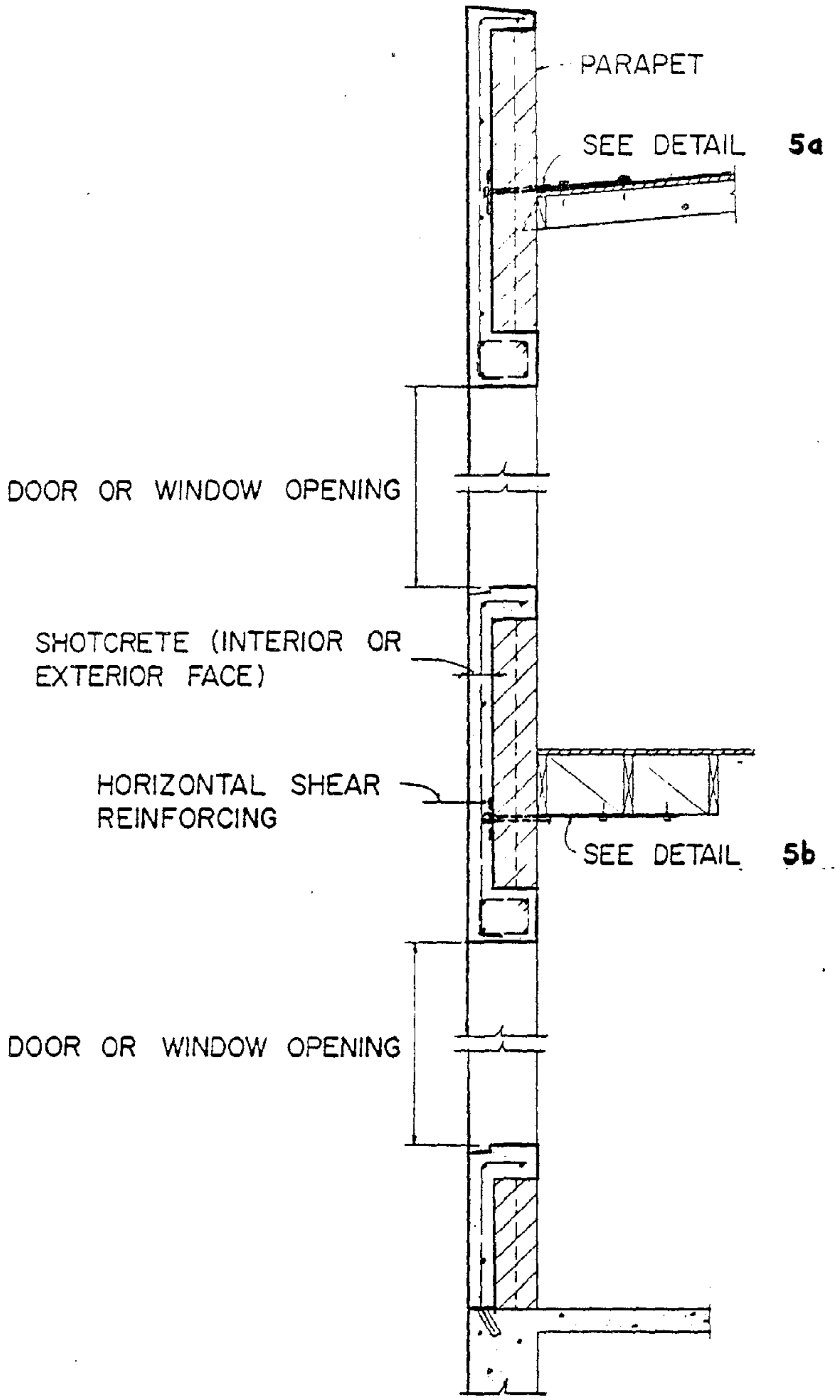
(b)



(c)

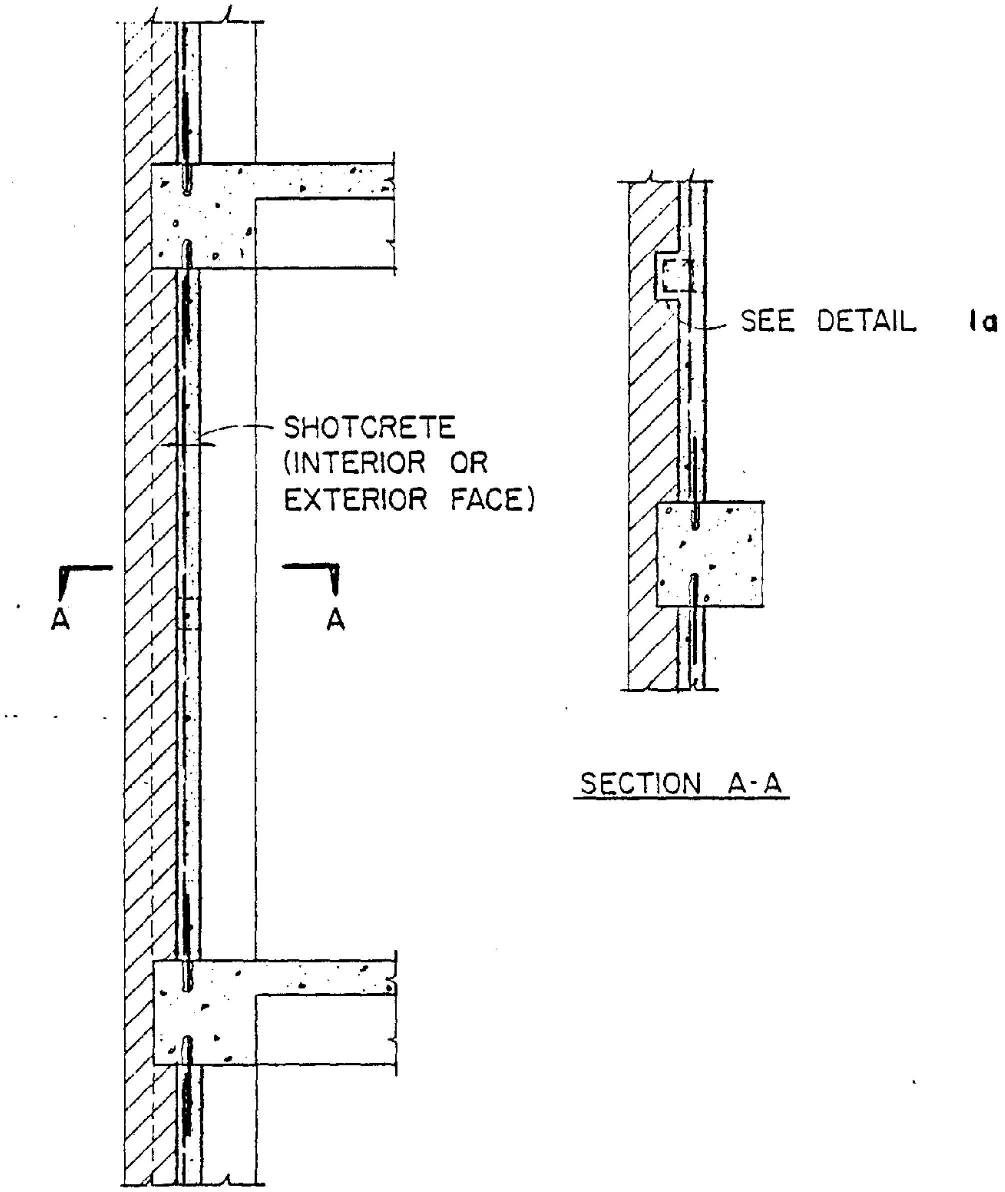
STRENGTHENING OF EXISTING
CONCRETE COLUMNS & BEAMS

FIGURE NO. 2



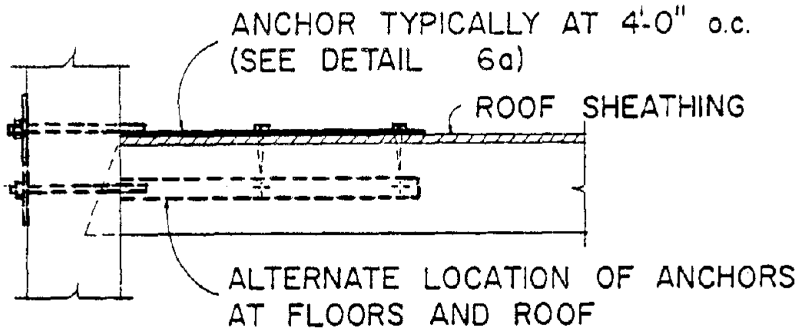
TYPICAL STRENGTHING OF MASONRY WALLS

FIGURE NO. 3

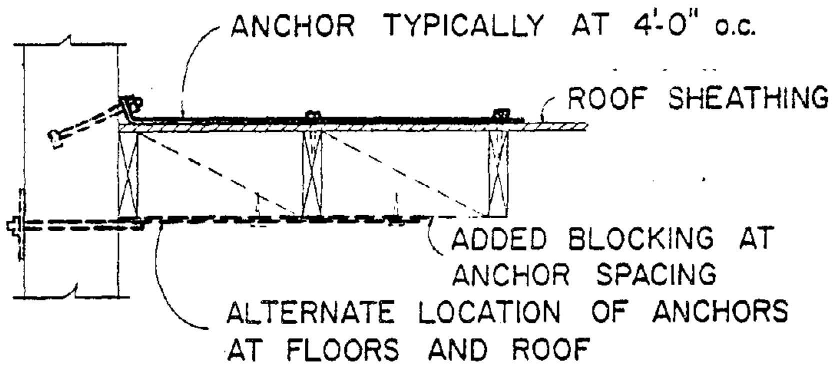


TYPICAL STRENGTHENING OF BRICK MASONRY WALLS

FIGURE NO. 4



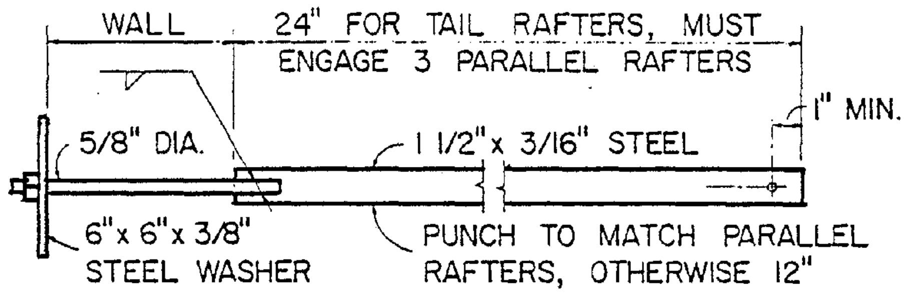
(a)



(b)

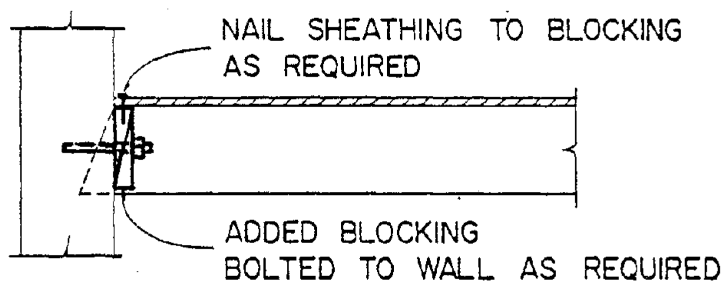
CONCRETE OR MASONRY WALL ANCHORS TO WOOD DIAPHRAGM

FIGURE NO. 5



ANCHOR DETAIL

(a)

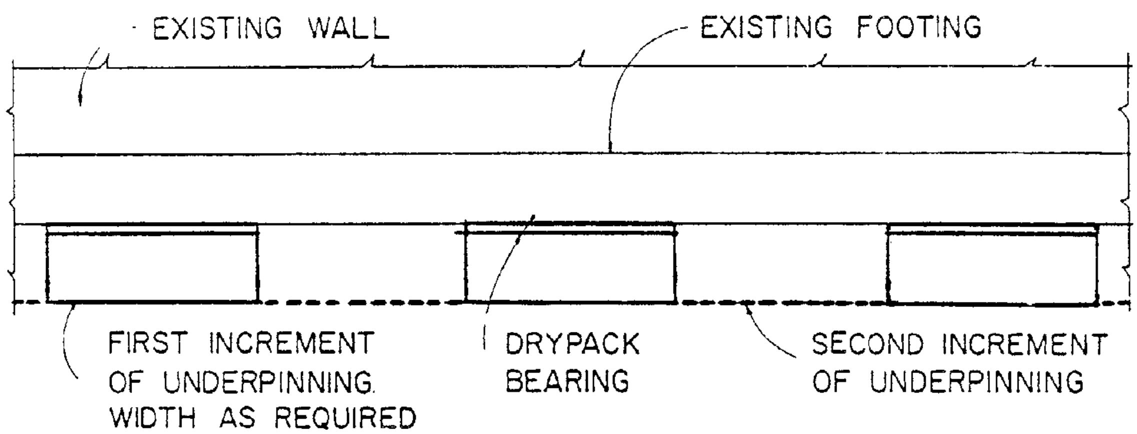


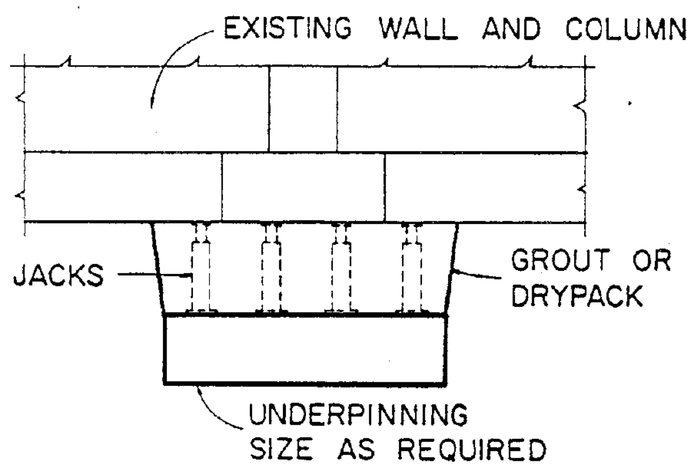
WALL SHEAR TRANSFER

(b)

STRAP ANCHOR AND BLOCKING DETAILS

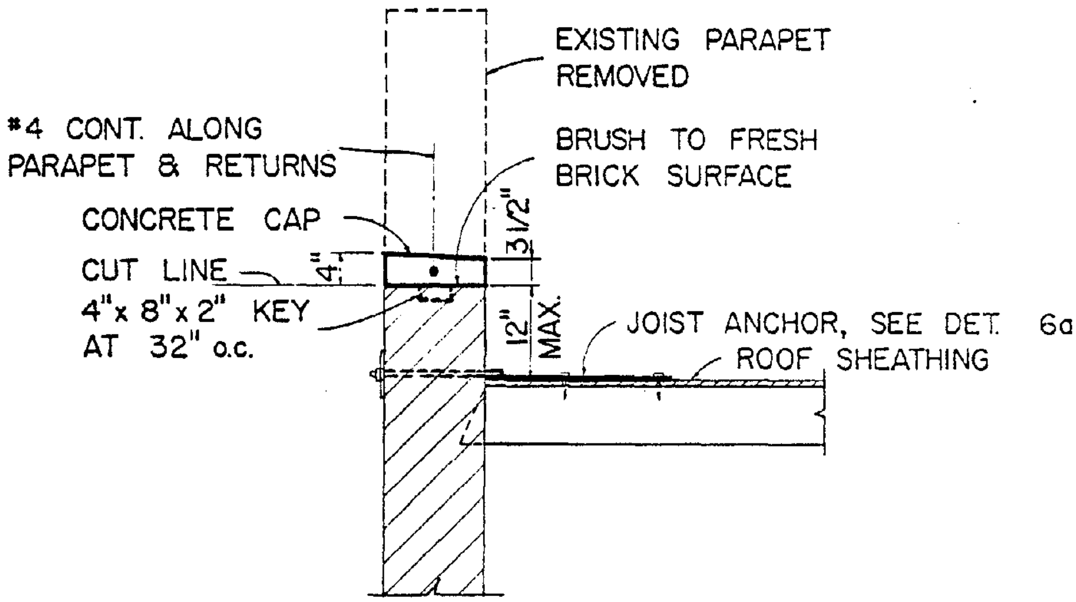
FIGURE NO. 6





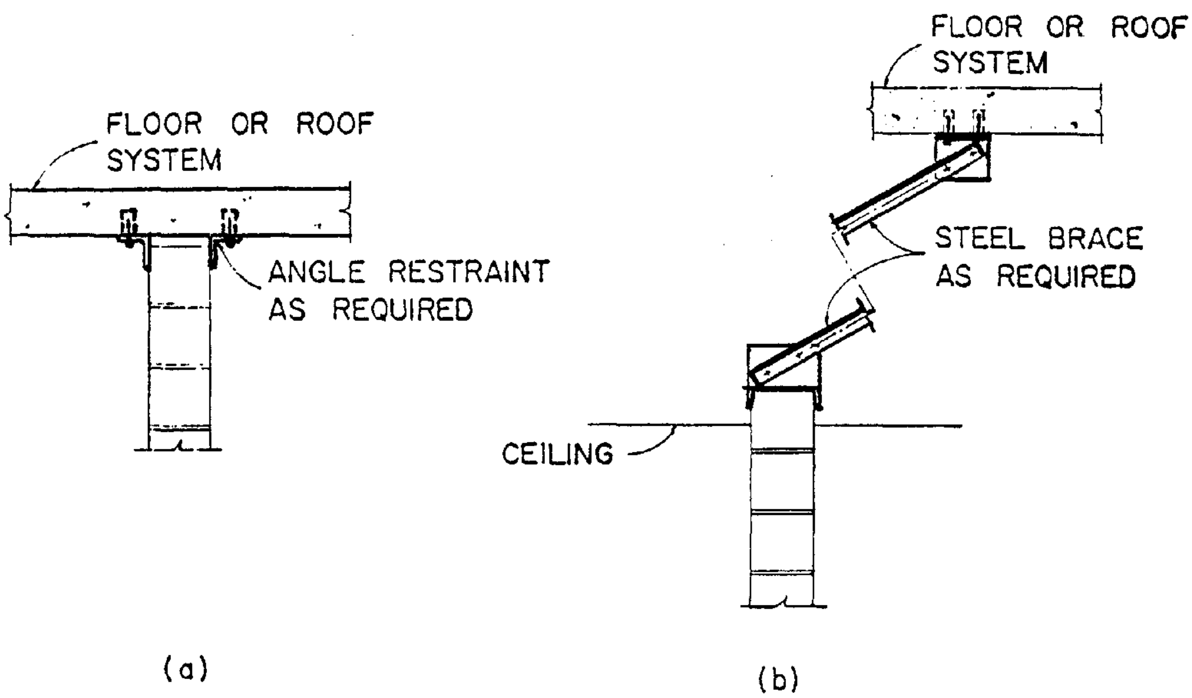
UNDERPINNING DETAILS

FIGURE NO. 7



TYPICAL PARAPET CORRECTION DETAIL

FIGURE NO. 8

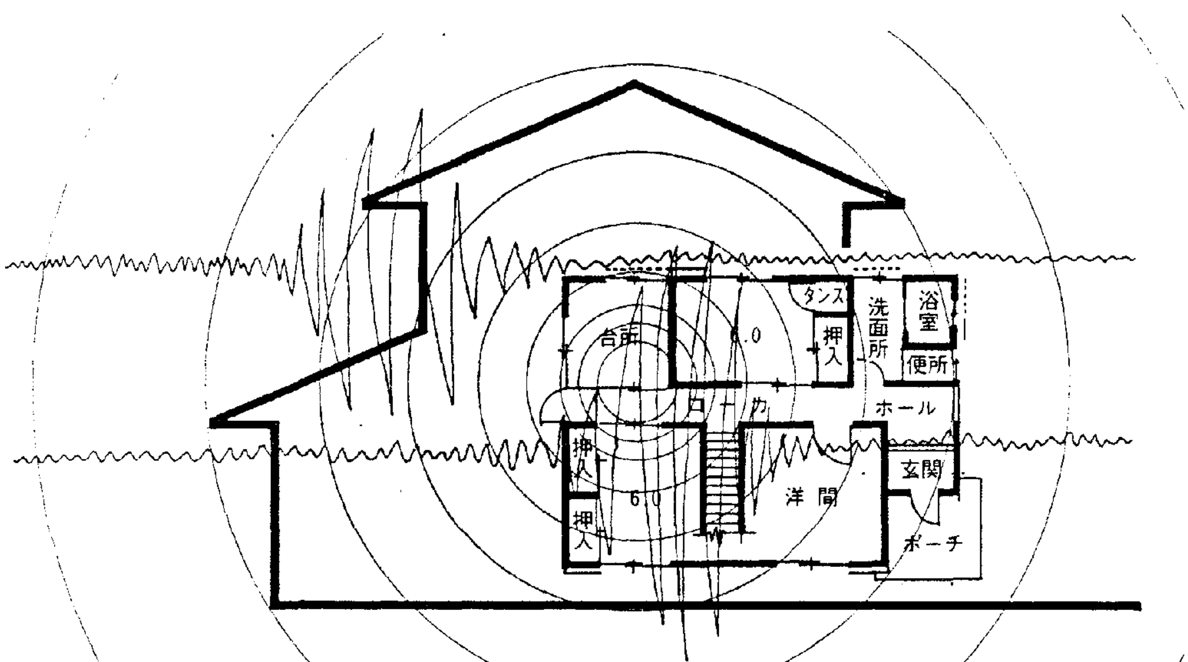


NON-BEARING PARTITION ANCHORAGE

FIGURE NO. 9

わが家の
耐震診断と補強

HOW TO EVALUATE THE SEISMIC SAFETY OF YOUR HOUSE
AND HOW TO RETROFIT IT



昭和53年３月

静岡県都市住宅部建築課

住宅の地震対策

最近、静岡県では、昭和49年5月の伊豆半島沖地震、昭和51年8月の河津地震、53年1月の伊豆大島近海地震等の被害地震が発生しており、また、学会において、東海地震説が発表される等地震に対する関心が急速に高まっております。

そこで、昭和53年3月から市町村、土木事務所、県庁建築課に建築相談窓口を設け県民のみなさんの住宅に関する地震対策の相談にあずかることとしました。

建築相談窓口の利用

建築相談窓口では、次の

1. 耐震診断のすすめ方及び補強工法の簡単な相談
2. 精密診断を実施する専門家（建築士、大工さん等）の紹介
3. 補強に要する工事費の融資等についての相談

におこたえしています。

わかりやすい自家診断

住宅の地震対策を進めるには、まず、自分の家がどの程度の耐震強度を持っているかを知らなければなりません。そこで、県では、誰でも計算できて、おおよその目安がつけられる自家診断法を作りました。

診断の手順

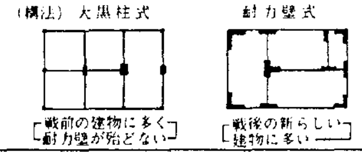
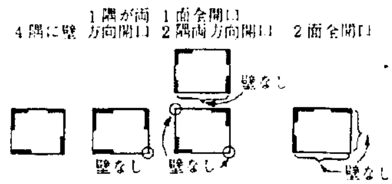
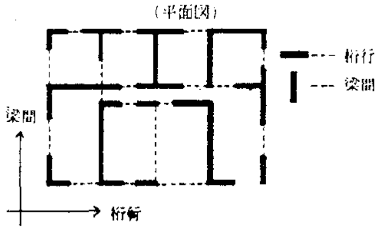
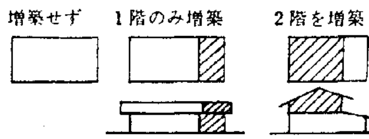
これは、過去の木造住宅の地震被害、破壊実験、建築基準法の耐震規定等を基にして作ったものであり、敷地の地盤、建物の構法、階数、屋根材、壁の配置、筋かいの有無等7つの指標から総合評点を計算し、強度の判定ができるようになっています。

この診断をするには、次の準備をして下さい。

1. 壁の配置がわかる平面図
2. 建築面積(坪)の計算(2階建の場合は、1階部分の建築面積)

以上の準備ができたら､次の｢自家耐震診断法｣の表に従ってあなたの住宅を自分で耐震診断してください。

自家耐震診断法

	項目				評点	解説
a	構法＼地盤	良い	普通	悪い		
	耐力壁式	1.2	1.0	0.9		
	大黒柱式	0.9	0.8	0.6		
b	階数＼屋根葺材	軽い	重い			重い屋根――瓦葺、カヤ葺等 軽い屋根――鉄板葺、スレート葺等
	平家	1.2	1.0			
	2階建	0.8	0.7			
c	4隅に壁	1.0				
	1隅が両方向とも開口	0.9				
	1面全開口、2隅が両方向開口	0.8				
	2面全面開口	0.7				
d	筋かい有り	1.5				建物のどこかに筋かいがあれば筋かい有りとみなす。
	筋かいなし	1.0				
e	見かけの壁率が0.05未満	0.2				この項目については、桁行方向、梁間方向別々に計算して壁の長さの少ない方で行う。 (平面図)  見かけの壁率＝壁の全長（間）／建築面積（坪） (壁の全長は外壁と内壁の合計とする。)
	0.05以上0.15未満	0.4				
	0.15 〃 0.25 〃	0.7				
	0.25 〃 0.35 〃	1.0				
	0.35 〃 0.45 〃	1.3				
	0.45 〃 0.55 〃	1.7				
	0.55 〃 0.65 〃	2.2				
	0.65以上	3.0				
f	増築せず	1.0				
	1階のみ増築	0.9				
	2階を増築	0.8				
g	老朽化していない	1.0				建物全体から判断し、特に北側の台所、風呂場等の土台、柱脚部分を診断する。
	腐蝕著しい	0.8				
総合評点	E＝a×b×c×d×e×f×g					

記入上の注意

1. 2階建の場合は、1階部分を診断する。
2. 評点の記入は、左欄の項目から該当する数値を選び出す。
3. 地盤について
   良い…岩盤、硬い砂礫層、砂利混りの層で硬くしまったところ
   悪い…田、沼地等を埋めたところ、腐植土、泥土、造成地等で盛土したところ、大雨のとき出水する低湿地のところ
   普通…上記以外のところ

診断結果の判定

前頁の表に掲げてあるａからｇまでの評点を選び出し、それを掛け合わせて得られた数値が高ければ高いほど良いのですが、診断結果は、次の表によって判定して下さい。

総合評点	判定	今後の対策
0.5未満	倒壊のおそれがきわめて高い。	専門家と補強方法について相談されるようおすすめします。
0.5以上 1.5未満	専門家の精密診断を要する。	専門家に現地精査による精密な診断をしてもらい、耐震強度を確認されるようおすすめします。
1.5以上	まず倒壊することはない。	特に対策は必要ないと思われます。

（注）
精密診断を行う専門家……県で行う研修を受けた建築士、建設業者、大工さん等
（建築相談窓口に研修を受けた専門家の名簿を備えてありますのでご利用下さい。）

精密診断

精密診断は、専門家が直接現地におもむき、建物の壁の質、柱･梁の継手、土台・柱の腐蝕程度等細部にわたって調査し、総合評点を算定するとともに、各部分について改善を要する事項を指摘するものです。

その費用は、建物の大きさにより異なりますが、大体３万円から６万円位です。

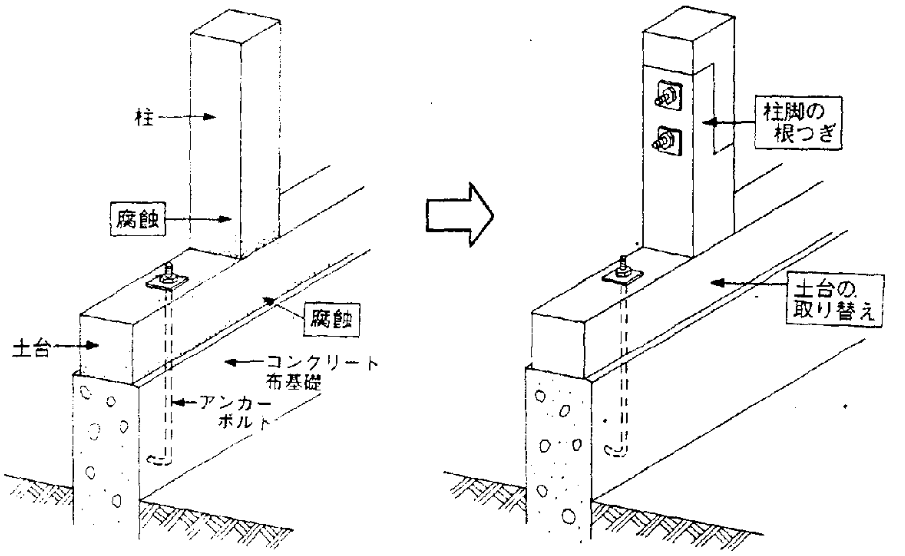
補強工法

木造住宅の構造は、土台、柱、梁等で骨組みを作り、適宜に土壁又は板張り壁を設置して、地震に抵抗するものとなっています。

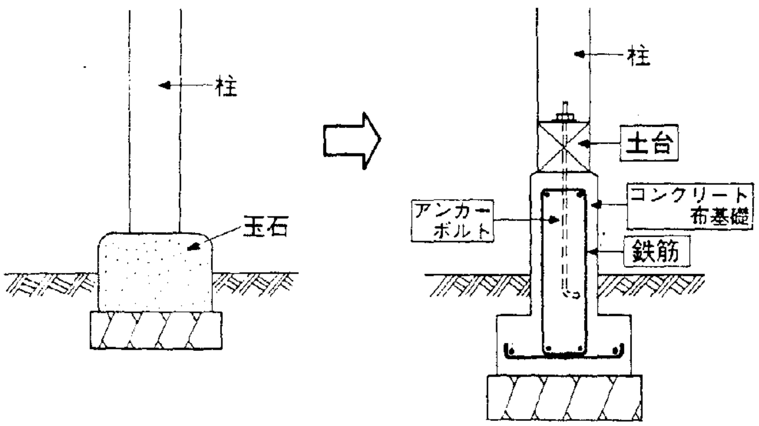
補強工事をするには、専門家と相談して建物の現況、予算等を考慮して、その住宅に最も適した方法を見出すことが大切ですが、特に、地震動を受けた場合でも、その建物がゆれないように剛性の高い構造とすることと、少し位ゆれても、柱、梁がバラバラにならないように接合部を金物等で補強することが必要です。

従って、補強工法の一例として次のようなものが考えられます。

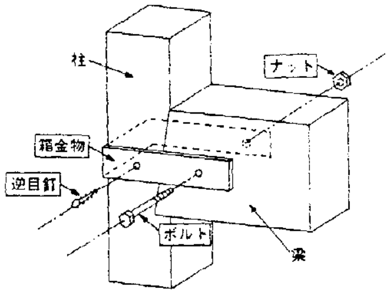
1．土台、柱、梁等で腐蝕している場合は、これを取替え又は修理する。

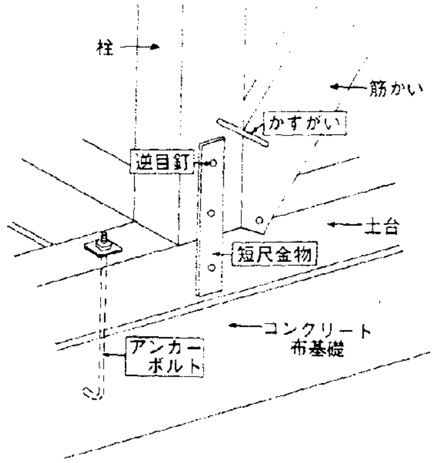


2．しっかりした基礎がない場合又は不規則に沈下している場合は、堅固な基礎を作り、これに土台をアンカーボルトで締める。

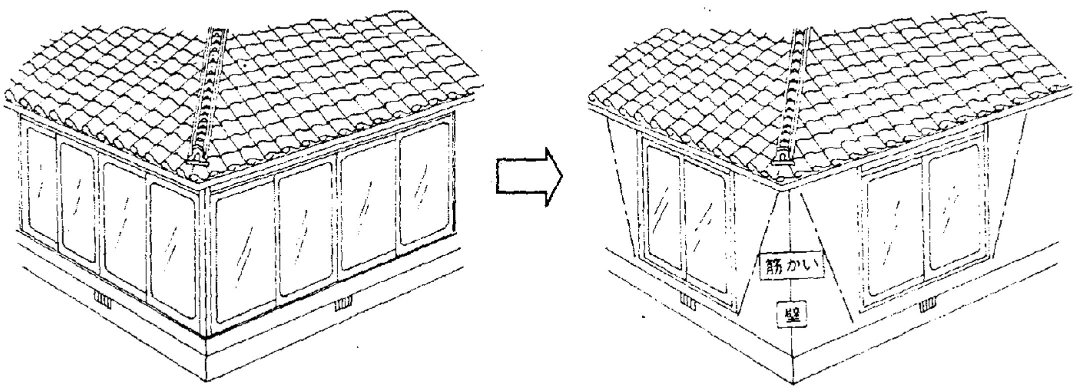


3．土台、柱、梁等の接合を金物等を使用して堅固にする。

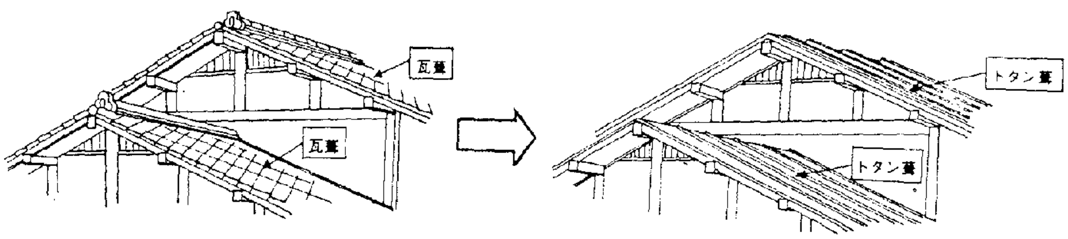




4．前後振動、左右振動に有効に抵抗するよう壁を適当に配置する。
（窓等を壁に替える。）



5．木造住宅の重量の大半は、屋根瓦とふき土であり、住宅にかかる地震力は、建物の重量に比例して増大するので、屋根を軽いものに替える。



融資制度

住宅を建て替える場合又は補強工事等をする場合には、住宅金融公庫と県による次のような融資制度があります。

申し込みの受付は、金融機関で取扱っております。

なお、住宅が著しく危険であり、県等から「改善通知」を受けることのできるものについては、優先的に融資が受けられます。

この場合は、最寄りの建築相談窓口にご相談下さい。

◎住宅の補強工事をする場合

▶住宅改良資金（住宅金融公庫）

(1)対象者……個人　　(2)年利率…… 6.0%

(3)貸付額（償還期間）……住宅改良工事費の70%以内の額で
10～140万円（10年以内）

◎住宅の建て替えをする場合

▶個人住宅融資（住宅金融公庫）

(1)対象者……個人　　(2)年利率…… 5.5%

(3)貸付額（償還期間）……

構造（償還期間）	貸付額
木造・組立木造（25年以内）	180～440万円
不燃構造（25年以内）	200～450万円
簡易耐火構造（30年以内）	240～490万円
耐火構造（35年以内）	240～490万円

（注） 償還期間は法律改正後53年度より適用される予定のものである。

▶静岡県個人住宅建設資金（静岡県）

(1)対象者……住宅金融公庫の融資を受ける者

(2)年利率…… 6.0%

(3)貸付額（償還期間）……200万円以内（15年以内）

（注） 融資金額等は、53年度の申込の場合であるが、地域によって若干の差があります。

これから建てようとする

木造住宅を耐震的なものにするための知識

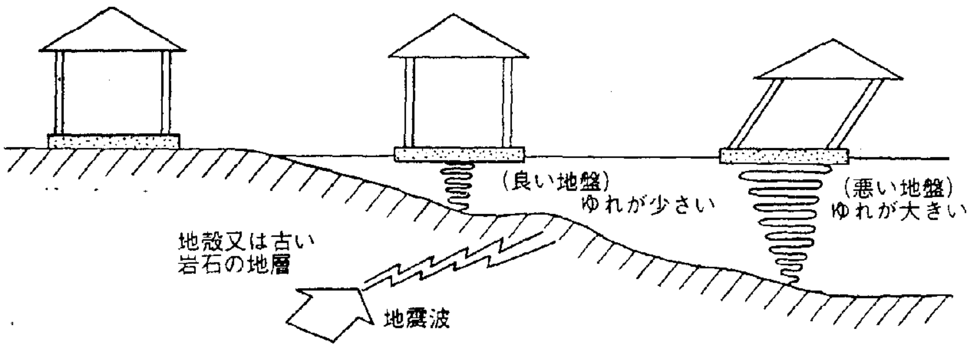
敷地の選定

(1)地盤…良い地盤の敷地を選ぶことが大切です。

①良い地盤…岩盤、硬い砂礫層、砂利混りの層で硬くしまったところ

②悪い地盤…田、沼地等を埋めたところ、腐植土、泥土、造成地等で盛土したところ、大雨のとき出水する低湿地のところ

悪い地盤は、良い地盤と比べるとゆれ方が大きく、建物を壊す力が大きい



(2)周囲の状況…がけ崩れ、土石流、津波、洪水等の恐れのある敷地は避けることが大切ですが、やむを得ない場合は、建物を鉄筋コンクリート造にするか又は鉄筋コンクリート造の床の高いピロティー等で造るようにして下さい。

基礎

基礎はコンクリートの布基礎とし、なるべく高くして下さい。

悪い地盤のところでは、必ず鉄筋を入れて下さい。(補強工法の２の図を参照)

腐蝕防止

床下の通気を良くするため、布基礎には、換気孔をなるべく多く設けて下さい。

特に、玄関、台所、風呂場等の周りは通気を良くすることが大切です。

また、土台、柱の根元、床組には、防腐と白蟻の害を防ぐための薬剤を塗布して下さい。

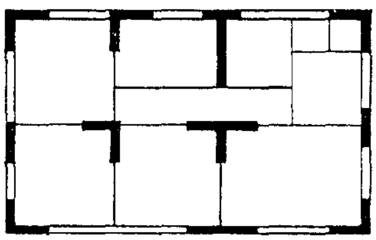
家の形

家の形は、地震の力が建物全体を平均に伝わるような単純な形が最も良く、凸凹の多い建物又は大きな部屋（24畳程度）のある建物は、地震に対して弱くなるので避けるようにして下さい。

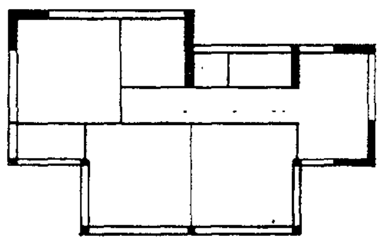
壁の量及び配置

壁は、多ければ多いほど良いのですが、建物全体につり合いよく、4隅に設けることが大切です。建物の一部にかたよって設けると、ねじれが生じて局部的に弱い個所ができ、被害がそこに集中する恐れがあります。

また、隅角部には壁を設けることが大切であり、壁のついてない独立した柱を設けることは極力避けて下さい。



良い



悪い

屋根

(1)屋根は、軽い葺材の方が地震には有利です。

(2)瓦葺には、他の葺材にない良いところ（遮音、断熱性、耐久性、見ばえ等）があります。従って、瓦屋根を選ぶ場合が多いのですが、瓦の重量は、建物の重量の大半を占めるので、できるだけ軽くするように工夫することが大切です。

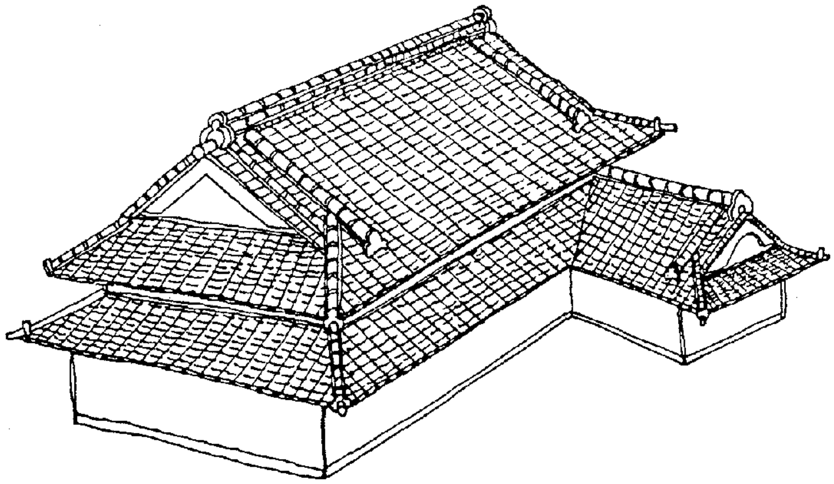
その例としては、次のようなことが考えられます。

①屋根全体の形を切妻等の単純なものにして、二重、三重造の屋根は避ける。

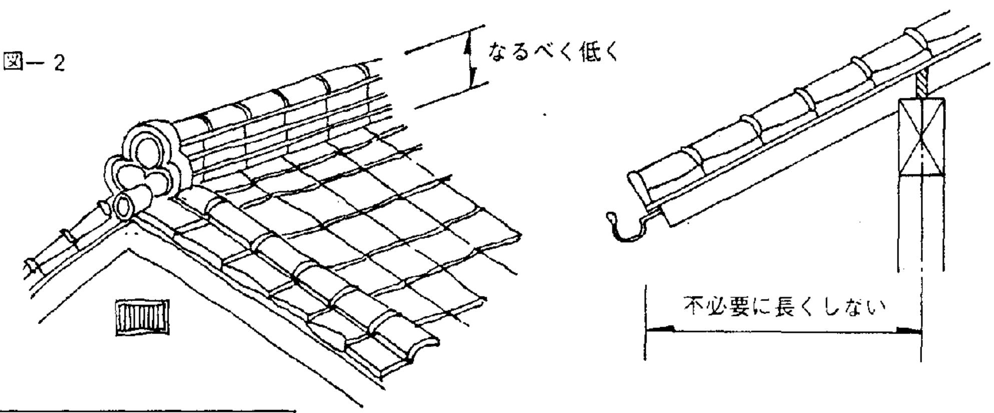
②入母屋造、寄せ棟造等の複雑なものはなるべく避ける。(図－1)

③棟瓦の高さはなるべく低くして、軒の出等も不必要に長くしない。(図－2)

図－1



図－2



金物の使用

補強工法のところでも述べましたが、建物がゆれても柱・梁・土台等がバラバラにならないように、柱・梁・土台等の接合部を金物で補強することが必要です。(補強工法の3の図を参照)

2階建ての場合

2階建ての場合は、間取りを考える時に、2階の柱が1階の柱の上にくるよう配置するとともに、建物の隅柱は、通し柱を設けるようにして下さい。

なお、詳細については、市町村建築担当課、土木事務所建築住宅課及び県庁建築課の建築相談窓口でご相談下さい。

INSPECTION AND RETROFITTING FOR EARTHQUAKE RESISTANCE VULNERABILITY OF HIGHWAY BRIDGES IN JAPAN

by Eiichi Kuribayashi
Osamu Ueda, Tadayuki Tazaki
Public Works Research Institute

SUMMARY

All highway bridges in Japan are supervised technologically through the authorized specifications by the Ministry of Construction. The ministry has conducted the inspection of highway bridges three times (i.e. 1971, 1976 and 1979). The first one in 1971 was to point out the deteriorated bridges liable to be damaged in earthquakes. The second in 1976 was to check the items being closely related with the possibility of damage. The third inspection in 1979 was to classify bridges according to their earthquake resistances. This paper introduces the procedure of the latest inspection and its retrofitting in 1979.

INTRODUCTION

It is necessary in earthquake disaster mitigation planning to extract structures liable to be damaged in earthquakes. Two methods exist for the extraction. The one is to point out the structures liable to be damaged when they have the vulnerably structural factors according to the experiences of past earthquakes. The other is to analyse structures and to judge their safety.

The inspection of highway bridges conducted by the Ministry of Construction, Japan, in 1979 sequentially applied both of the methods. Possibly vulnerable bridges were extracted by the former method. The vulnerable factors considered were;

(1) the design based on the old specifications,
(2) deteriorated materials, and
(3) vulnerable types of structures according to the damage in past earthquakes.

The extracted bridges were inspected by the latter method.

The priority of retrofitting was determined by the importance of bridges.

PROCEDURE OF THE INSPECTION

The inspection of highway bridges conducted in 1979 consists of four steps. The first step is to select the routes to be inspected, which are indispensable in emergency.

The second step is to extract the possibly vulnerable bridges. Referring the reports of past earthquakes, damage of bridges is more affected by the vulnerable subgrounds and substructures than superstructures, so that the formers are emphatically inspected.

The bridges extracted by the second step are to proceed to the third step. It is to inspect the stability of subgrounds and foundations, and

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I) Senior Research Engineer, Earthquake Engineering Division, Public Works Research Institute, Ministry of Construction, Tsukuba Science City 305, JAPAN

the section moduli of piers.

The forth step is to analyse structures dynamically if required.

The retrofitting method for each type of vulnerability identified is lastly proposed. The priority of retrofitting is to be determined by the availability of substitutive routes and the easiness of traffic resumption in emergency.

The procedure of the inspection is shown in Fig. - 1. This paper mainly describes the second and third steps.

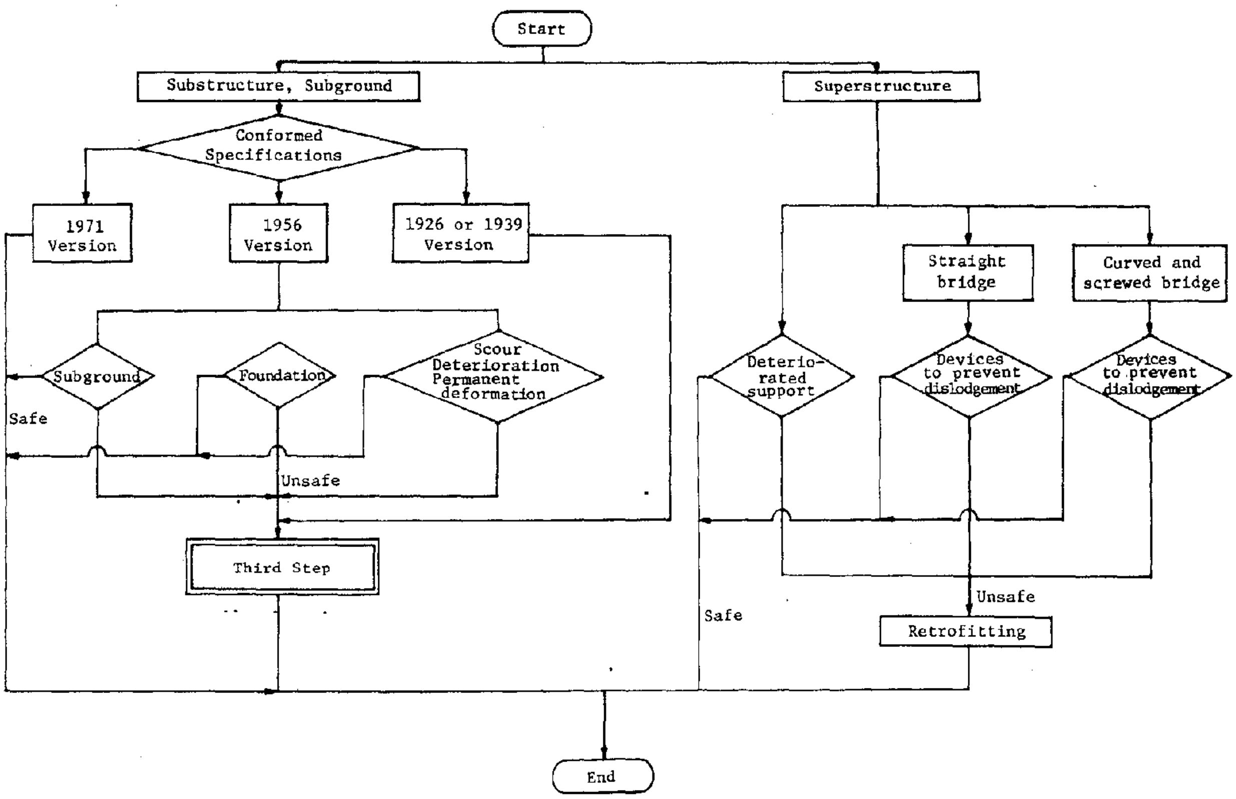

Fig. 1 Flow Chart of Inspection in the Second Step

Possibly Vulnerable Bridges (Second Step)

The second step is to extract the possibly vulnerable bridges which should proceed to the more detailed inspection in the third step. The vulnerable factors considered are as follows:

(1) Specifications Conformed

Owing to the progress of earthquake engineering, specifications have been revised several times.

At least the structures conformed with the latest specifications of 1971 were considered to have enough safety. The structures before the 1956 specifications were considered to be possibly unsafe. Those between 1956 and 1971 were judged depending on the subground, foundation and deterioration of the substructure.

For instance of the improvement of the specifications no attension had been paid to liquefaction before the specifications of 1971 were issued.

(2) Subground

a. Loose and Saturated Sand

Loose and saturated sand is liable to liquefy in earthquakes. Sandy layers which were less than 10 m deep and whose N-values were less than or equal 10, or the sites where historical liquefaction was reported were extracted.

b. Poor Subsoil

Peat layers or the sites where adjacent dikes and embankments settled were extracted.

(3) Substructure

a. Lack of Enough Rigidity

The substructure as shown in Fig. 2 suffered damage in Miyagiken-oki earthquake of 1978. The damage would have been attributable to the independent two caisons and insufficient rigidity of the tying members.

The pile bent substructure as shown in Fig. 3 experienced damage in Niigata earthquake of 1964.

Both types of the foregoing substructures have insufficient rigidity. Therefore the substructures without enough rigidity were extracted.

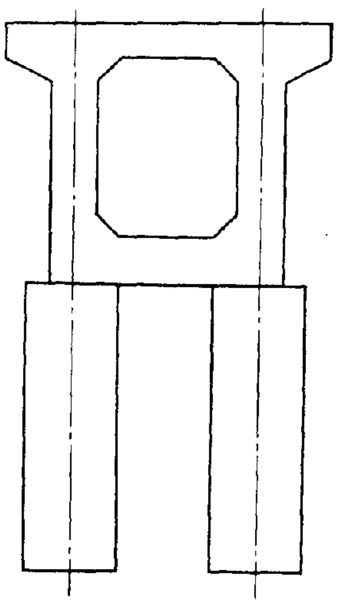

Fig. 2 Independent Caison Foundation

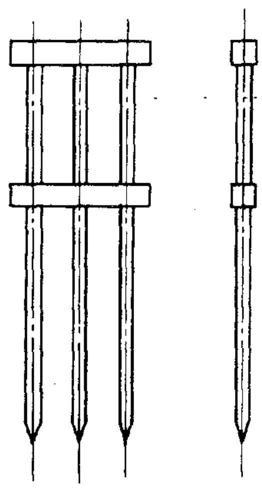

Fig. 3 Pile-bent Foundation

b. Brittle Materials

Substructures made of plain concrete, brick and masonry were extracted.

c. Settlement and Inclination

The substructures which settled or inclined were extracted.

(4) Superstructure

a. Curved Bridge

A curved bridge acts rather different in an earthquake from what is expected in conventional design. In conventional design a bridge was designed in longitudinal and transverse directions. However a curved bridge bears not only the forgoing loadings but also torsional loading. The curved bridges without considerations of torsional loading and whose radius were less than 100 m were extracted.

b. Skew Bridge

By the similar reason as curved bridges, skew bridges of less than 60° of angles were extracted.

c. Deteriorated Supports

The supports of deteriorated anchor bolts, deteriorated bearings and over-dislodged supports were extracted.

d. Lack of Devices to Prevent Dislodgement

The supports without devices to prevent dislodgements which were specified by the specifications of 1971 were extracted.

Classifying Bridges by their Resistance (Third Step)

The bridges extracted by the second step were to be inspected in the third step. Here only subgrounds and substructures were inspected, because superstructures do not affect the damage according to the experiences of past earthquakes as far as they passed the second step inspection.

(1) Subground

a. Liquefaction Resistance Factor

Liquefaction resistance factor, F_L is defined as the ratio of the resistance index of soil elements to dynamic loads R, and the shearing stress loads index to soil elements induced by earthquake motions L. The procedures to calculate R and L are shown in Reference [2]. Subground having the total thickness of the layers of greater than 10 m whose F_L were less than 0.6 was judged to be liquefied in earthquakes.

b. Bearing Capacities

In the relationship between the overturning moment and the bearing capacity of foundation three zones were defined as safe (A), slightly unsafe (B) and unsafe (C) in Fig. 4.

(2) Substructure

a. Section Modulus of Pier

Aged piers possibly have the insufficient section moduli compared to the current specifications. The section moduli of inspected piers were compared with those of the Sandard Design issued by the Ministry of Construction and other institutions, which were designed based on the current specifications. The checking charts are shown in Figs. 5 - 7. Fig. 5, Fig. 6 and Fig. 7 correspond to wall pier, column pier and rigid frame pier respectively. The line dividing zones A and B was drawn by enveloping the dimensions designed by the Standard Design. The line dividing B and C was drawn by multiplying by 1.1 (reserve strength of reinforcement) of the line between A and B. Zone C was determined to be preferentially retrofitted.

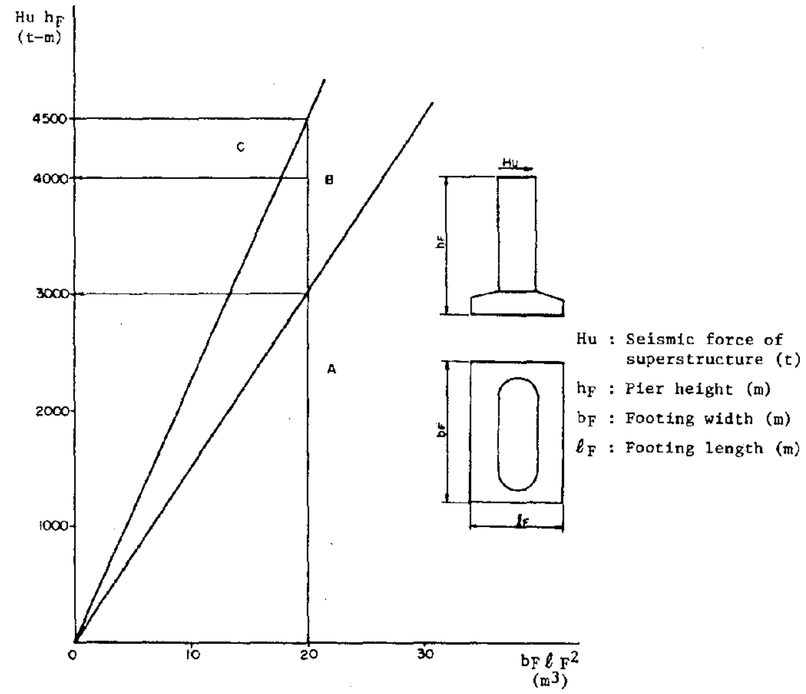

Fig. 4 Checking Chart for Bearing Capacity

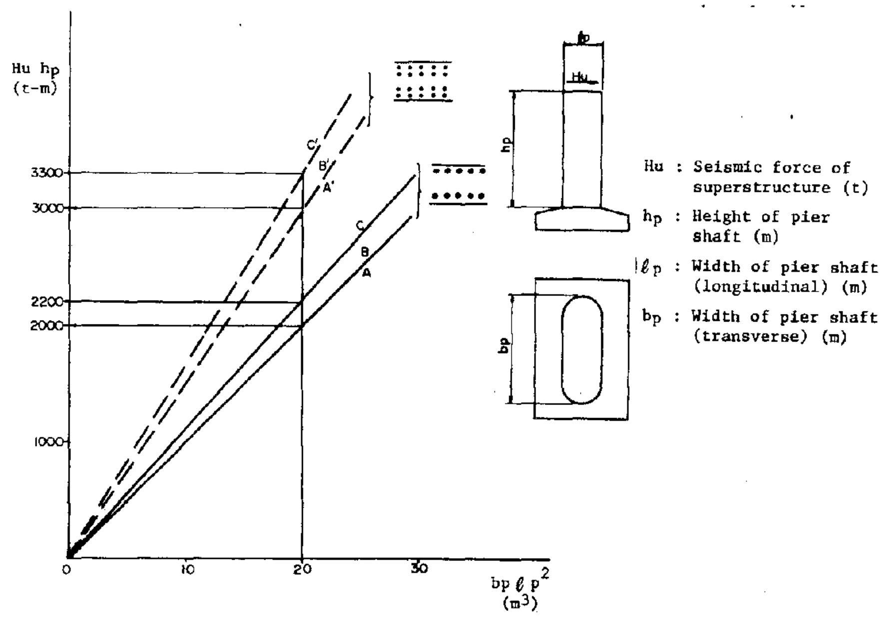

Fig. 5 Checking Chart for Wall Pier

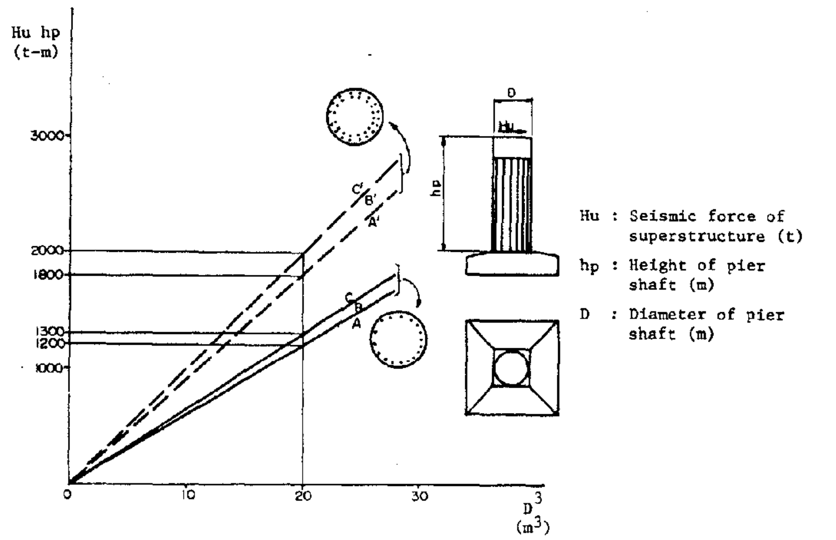

Fig. 6 Checking Chart for Column Pier

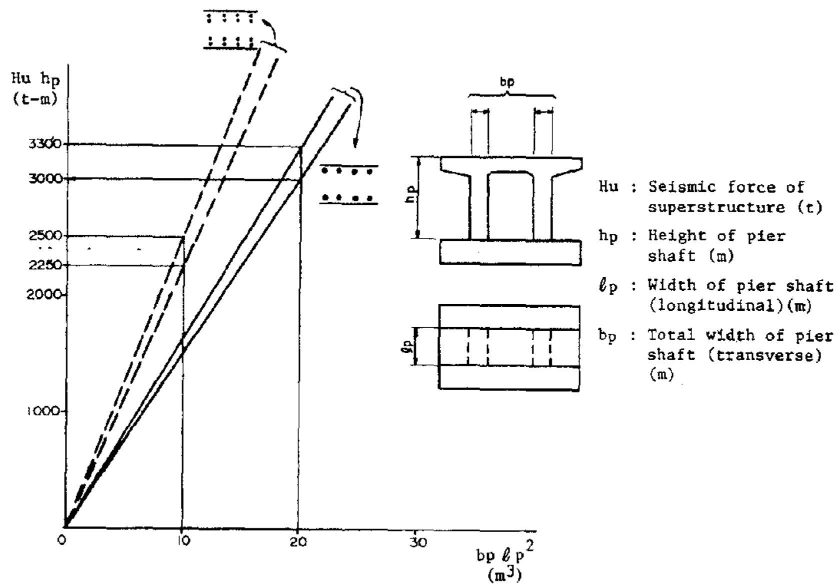

Fig. 7 Checking Chart for Rigid Frame Pier

b. Safety Factor of Pile Foundation

Aged pile foundations are liable to be damaged than other types of foundations according to the experiences of past earthquakes. The reason for this would be that there did not exist capable pile drivers in old days. Additionally most piles before 1950 were made of timber. Therefore pile foundation was exceptionally inspected by calculating the safety factor SF as follows.

$$SF = \frac{R_u}{V_i}$$

R_u : Ultimate bearing capacity of a pile (t)

V_i : Vertical reaction of pile i (t) $= \frac{V}{n} + \frac{Ve}{X_i^2} X_i$

V : Vertical load (t)

n : Number of piles

e : Eccentricity (m)

X_i : X coodinate of i-th pile (m)

Dynamic Analysis

The bridges extracted by the above step were inspected by applying the the dynamic analysis, if required.

Determining the Method of Retrofitting

The retrofitting method for each type of vulnerability identified is shown in Table 1.

Table 1. Proposed Method of Retrofitting

Classification	Vulnerable Factor	Method of Retrofitting
Subground		Surrounding by sheet piles Pile driving behind abutments Driving additional piles Sand compaction piles
Substructure	Scour Lack of enough rigidity Section modulus of pier Section modulus of footing Safety factor of pile foundation	Consolidation of foundation Additional rigidity Additional section Expansion of footing Additional piles
Superstructure	Curved bridge Screw bridge Deteriorated support Lack of devices to prevent dislodgement	Devices to prevent dislodgement Enlargement of bridge seat Connecting devices of adjoining girders Exchange of support Installing devices

DISCUSSIONS

About 37000 bridges were inspected in which 42% were judged to be retrofitted.

It is necessary to get a reasonable level of restrofitting from an economic point of view. Because of the low recurrence of damaging earthquakes, the retrofitting investment is obliged to be at a lower level, when the direct effects of retrofitting are only considered. However the retrofitting also has the indirect effects, such as the traffic and transportation, regional economy and opportunity loss for repair and reconstruction. When such indirect effects are considered, more retrofittings are reasonalized.

ACKNOWLEDGEMENTS

The author wishes to express his gratitude for the suggestions and supports of Messrs. Tatsuya Fujii and Yoshihiko Enami of Ministry of Construction, Japan.

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2) T. Iwasaki, et al : A Practical Method for Assessing Soil Liquefaction Potential Based on Case Studies at Various Sites in Japan, Second International Conference on Microzonation, San Francisco, 1978
3) A. Longinow, et al : Bridge Retrofitting Selection of Critical Bridges in a Road Network, Technical Council on Lifeline Earthquake Engineering Specialty Conference, Los Angeles, August 1977

RETROFITTING BRIDGES TO INCREASE THEIR SEISMIC RESISTANCE

Oris H. Degenkolb

The 1971 San Fernando, California earthquake was a major milestone in the seismic design of bridges. It was the first earthquake that shook modern type bridges and caused major damage to bridges in the contiguous forty-eight states. That event pointed out a number of deficiencies in bridge design specifications and practices that were in use at that time. Consequently, it was realized that a great number of existing bridges could be severly damaged or collapsed if subjected to earthquakes that could possibly occur during the life of the bridge. Severe damage or collapse of these bridges could be hazardous and cause serious disruptions to lifelines and badly needed transportation routes at a time when they are urgently needed.

One of the major seismic deficiencies of pre-1971 bridges is that superstructure units were not adequately connected at hinges and bearings. Severe shaking could cause spans to drop off of their supports, as illustrated in Figure 1.

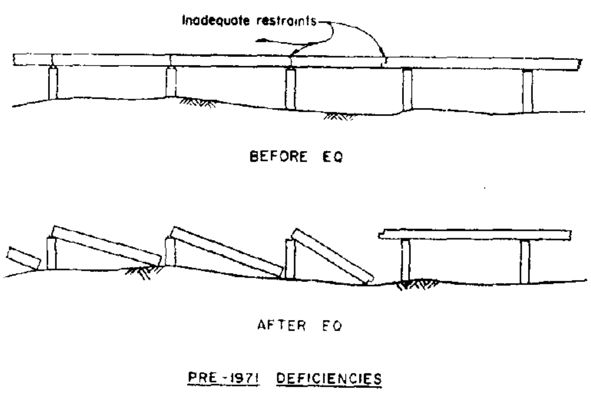


Figure 1

Figure 2(a) shows a number of other common structural seismic deficiencies that were responsible for the bridge failures in the 1971 San Fernando earthquake. Figure 2(b) notes the changes that were made in bridge design practice to make new bridges more seismically resistant. It is obvious that all of these improvements cannot be made to existing bridges and it is generally not practical to increase the seismic resistance of older bridges to the level achieved with new construction.

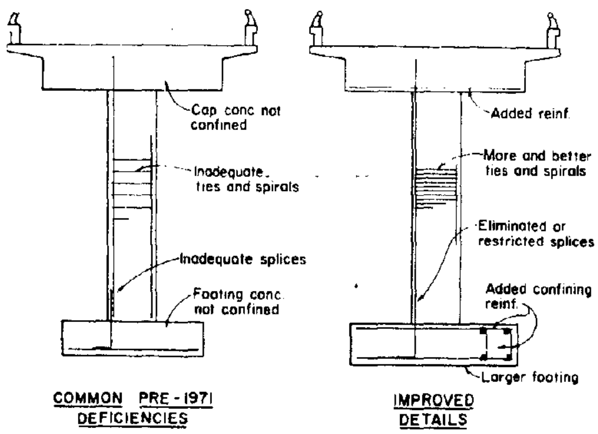
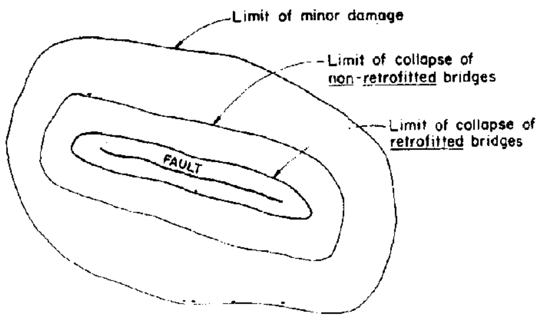


Figure 2

Fortunately, connecting segments of a structure together to alleviate the deficiency illustrated in Figure 1 is the least costly but the most effective method of retrofitting older bridges. This is generally accomplished by connecting the bridge segments together with restrainers consisting of steel cables or rods. Although this strengthening will not overcome all of the other deficiencies it will, in many instances, minimize them to some extent.

A certain amount of damage can be expected in seismically shaken bridges regardless of whether or not they have been retrofitted with hinge and bearing restrainers. This type of damage will usually consist of cracked abutment wingwalls; damage to girder bearings and grout pads; crushed ends of railings, curbs and sidewalks at joints; spalling of decks at joints; minor lateral displacement of decks at joints (especially in skewed spans); and spalling of concrete columns. This damage will occur because restrainers must permit some movement required for normal functioning of the bridge and, when acting during an earthquake, will permit additional differential movement of the structural units.

As illustrated in Figure 3, the area surrounding the fault within which bridges might collapse should be diminished considerably if the bridges are retrofitted with hinge and bearing restrainers.



EXTENT OF BRIDGE DAMAGE
CAUSED BY A
MAJOR EARTHQUAKE

Figure 3

Many older bridges provide very little or no restraint for keeping the superstructure seated on the abutments. Figures 4(a) and (b) show how superstructures can be shaken off of their supports if bearings, shear keys, or columns fail or permit excessive movements.

One of the more common seismic deficiencies is shown in Figure 4(c). Joints at the ends of simple spans and intermediate expansion joints in long continuous bridges permit the bridge to act as a number of individual units when shaken by an earthquake. Even moderate earthquakes may damage the bearings and joints in the decks, curbs, and railings of these structures. More severe or longer duration earthquakes can fail bearings and cause excessive forces and deflections in the columns, leading to collapse. If these joints could be connected to make them act as single units, column forces and deflections would be reduced considerably -- increasing the level of seismic resistance of the entire bridge. Unfortunately, restrainers at these joints must usually be left with enough slack in them to accommodate normal daily and seasonal movements.

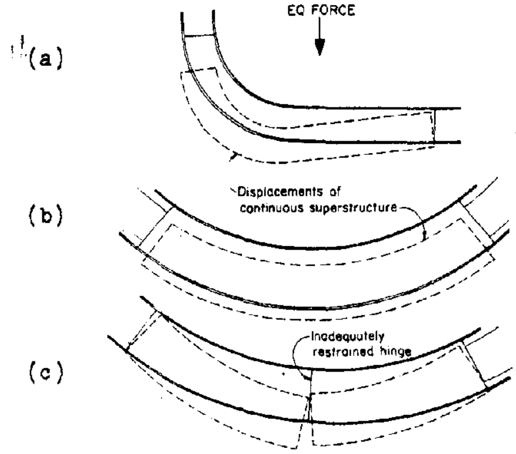
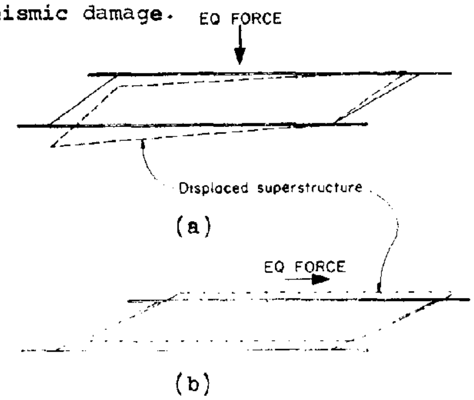


Figure 4

EFFECT OF EQ FORCES
ON CURVED BRIDGES

Skewed bridges are generally much more susceptible to earthquake damage than a similar size and type square bridge. Figures 5(a) and (b) illustrate the additional actions involved. In addition to the general lack of adequate transverse restraint provided at the abutments of older bridges, skews complicate the details and increase the structure's vulnerability to seismic damage.



EFFECT OF EQ FORCES
ON SKEWED BRIDGES

Figure 5

The two most commonly used materials for hinge and bearing restrainers in California are 3/4" 4x19 steel cable (Federal Spec. RR-W-410c) and 1¼" Ø high strength steel bars (ASTM A-722 with supplementary requirements).

Figure 6 shows the results of tests in which cables and bars were tensioned from near zero stress to specified minimum yield stress (assumed to be 0.85 F_u for cables) for 14 cycles and then pulled to failure. Figure 7 shows the results of tests in which cables and bars were stretched to failure but the loads were reduced to nearly zero at approximately one inch increments of elongation. The gage length of all specimens was 114 inches.

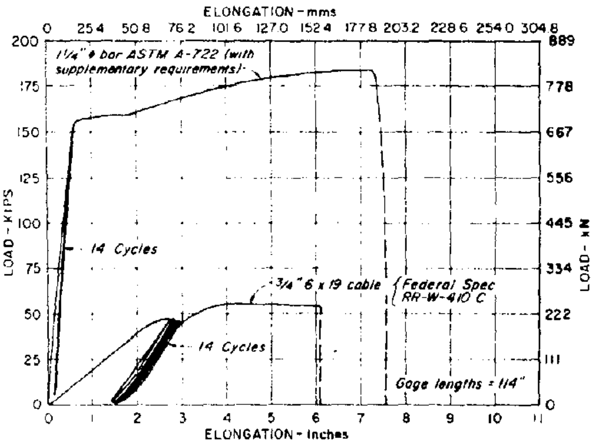


Figure 6

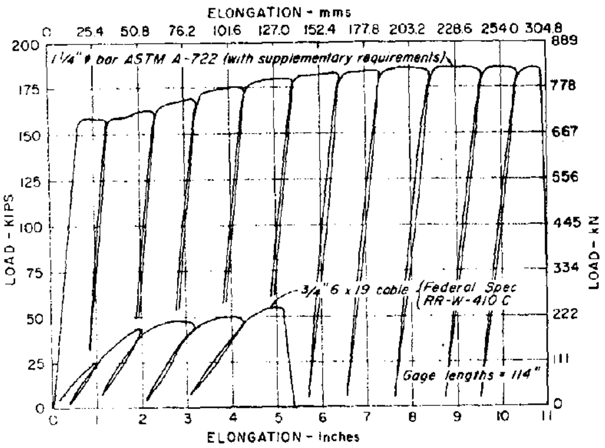


Figure 7

Cycling 3/4" cables within the elastic range requires more than twice the amount of energy than cycling an equivalent number of 1¼" Ø bars of the same length for the same number of cycles. This is due to the fact that bars have a greater modulus of elasticity and the elongation within the elastic limit is less than for cables. Within this range the cables and bars store energy but do not dissipate any significant amount.

When stretched beyond their elastic limits, bars dissipate approximately 3 times as much energy as the equivalent number of the same length cables.

If restrainers are permitted to yield, greater joint openings and column deflections will be realized. Once either type restrainer is stretched beyond its elastic limit it obviously will not assist in closing the joint to its normal position. Although bars will dissipate more energy than cables when failure occurs, the elongation will also be much greater. This could be an extra factor of safety in some structures but could be disastrous for structures with relatively short, stiff columns. When a restrainer is stretched to its ultimate limit, however, the structure is vulnerable to any additional shocks.

Considering the impreciseness of predicting a bridge's response to a possible future earthquake, it is generally not prudent to depend on restrainers acting beyond their elastic limit.

Restrainers should be capable of developing a minimum force equal to 25% of the weight of the lighter segment of superstructure connected, if Working Strength Design methods are used. When using Load Factor Design methods and the yield strength of the restrainer material, almost identical results are obtained by using 33% of the Dead Load. Column shears should be ignored in either case. Larger restrainer capabilities should be provided whenever required by dynamic analysis. A minimum of two restrainers are used at each bent or hinge -- one as close as possible to each edge of the superstructure. Restrainers are adjusted to permit normal movements of the joint and to start acting as soon as maximum normal open joint width is exceeded.

Assuptions concerning the interaction between the bridge and earth at the abutments is one of the greatest uncertainties in making a dynamic analysis. For this reason the minimum amount of restraining force may be satisfactory for many relatively short square bridges with only one hinge or joint. However, geological conditions, seismicity of the site and structural features may require that greater restraining forces be provided. Dynamic analyses will generally indicate whether cables or rods are preferred for any particular installation.

Slightly different assumptions for restrainer arrangements, foundation conditions, column stiffnesses, abutment restraints linear or non-linear action of the restrainers and columns, etc. can make drastic differences in the results of a dynamic analysis. In some cases the computor has given forces in restrainers that were so low, or movements of joints so little, that they did not appear to be consistant with observed actions of structures.

Considering all of the uncertainties and assumptions that are made in making a dynamic analysis, it is realized that the seismic design of a bridge is a developing art rather than an exact science. A number of analyses should be made and the results tempered with judgement.

The ideal restrainer should absorb and dissipate energy, keep joint movements within a safe range, and force the structure back to its pre-earthquake position. For practical reasons, the most suitable devices for new construction are not necessarily the best for retrofitting existing bridges Most of our restrainers to date have used steel cables or rods which act as tension members only.

These devices may not be ideal from a strictly theoretical point of view and they may not prevent as much damage as other types of restrainers that have been considered but, reviewing all of the factors involved, they are hard to beat They will raise the level of seismic resistance of a bridge, they are relatively easy to install and they are economical.

One of the problems of adding restrainers to existing bridges is attaching to existing members. Existing construction often is not strong enough to develop the required anchorage forces. In these cases, existing features may have to be strengthened in order to prevent premature failures. Another problem is that restrainer forces, if fully developed, may fail the columns or other portions of the bridge. In spite of these problems, restrainers by themselves can decrease a bridge's vulnerability to damage more than any other retrofitting system. The most seismic protection can be obtained for the least money by retrofitting existing bridges with restrainers. In the meantime, studies are being made for possibly retrofitting columns and footings of selected structures sometime in the future.

Figure 8.

Detail for restraining hinges of T-Beam bridges.

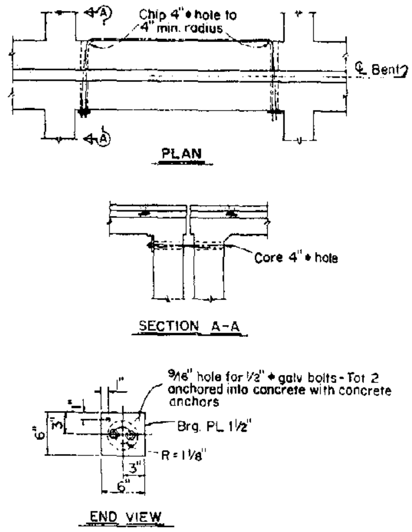


Figure 9.

Most commonly used detail for retrofitting concrete box girder bridges. Concrete bolsters prevent cable anchorages from destroying existing diaphragms.

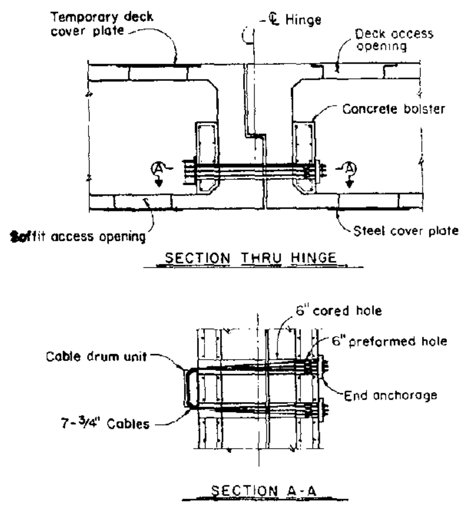


Figure 10.

Seven cables passed through a hinge joint three times to give the restraining force of 21 cables.

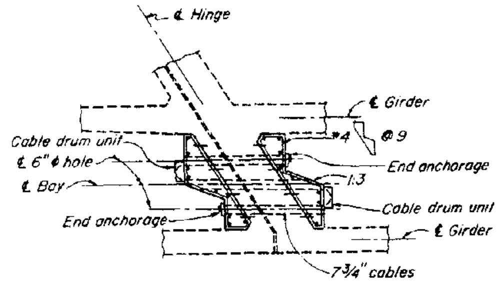
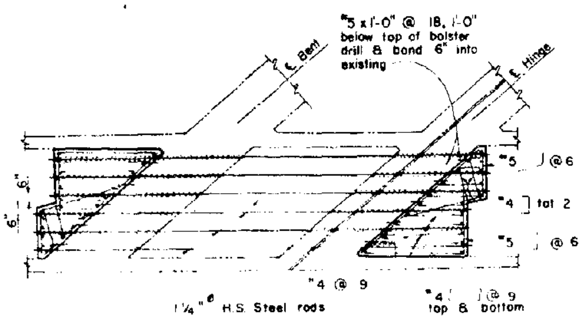


Figure 13.

Plan view of steel rod restrainers similar to Figure 11. Bolsters used to compensate for skew and strengthen hinge diaphragm.



Firuge 11.

High strength steel rods used to limit hinge movements. Long rods are used to absorb energy.

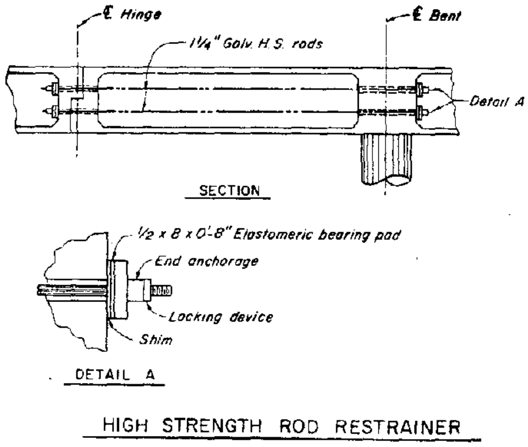


Figure 12

Detail of transverse restrainer used in conjunction with high strength steel rod restrainers to limit differential transverse movements of hinges.

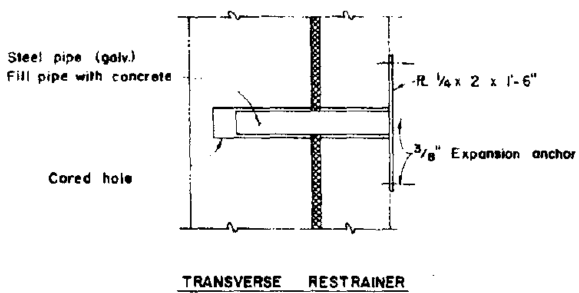


Figure 14.

Simple restrainer for precast - prestressed I girder spans at inverted T-bent.

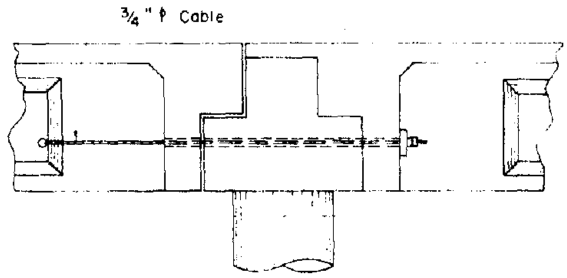


Figure 15.

Restrainers for short drop-in spans. Longer spans require more cables in order to limit amount of hinge movements.

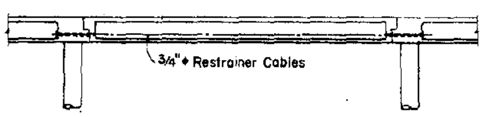


Figure 16.

Restrainer for limiting hinge movement of suspended slab span.

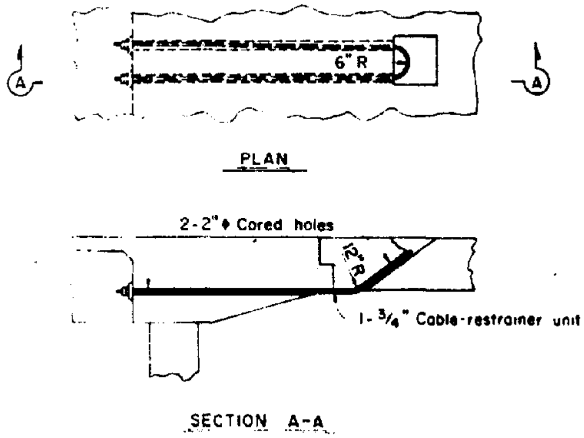


Figure 18.

3/4" cables used to connect steel girders supported on a steel bent cap.

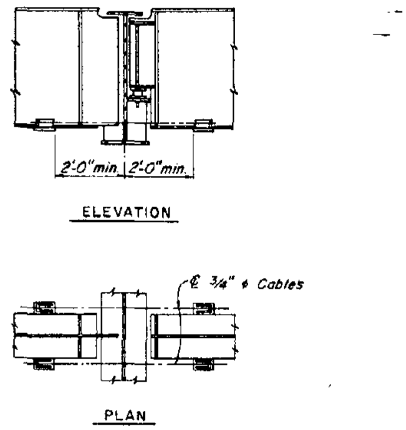


Figure 17.

Typical restrainers for connecting steel girders to concrete bent caps.

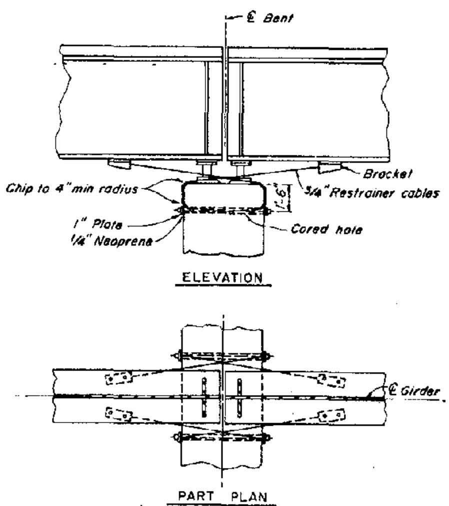
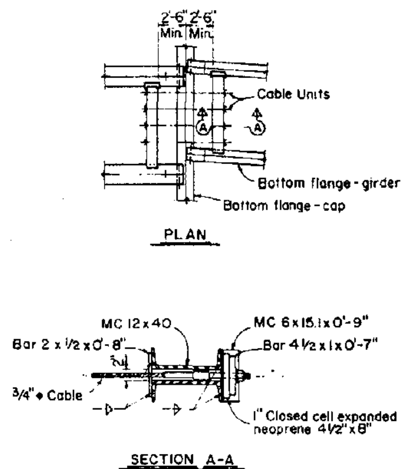


Figure 19.

Typical restrainers for steel girders supported on steel bent caps where girders in adjacent spans are offset.



REPAIR AND RETROFIT WORKS FOR EXISTING HIGHWAY BRIDGES

Tatsuo Asama, Yukitake Shioi,
Tadayuki Tazaki and Hideya Asanuma

Public Works Research Institute
Ministry of Construction

I. Introduction

Japan has an extensive road network consisting of 40,000 km of national highways and 130,000 km of prefectural roads; they account for 87% of the total domestic transport volume in tonnage, or 37% of the total ton-km.

If a major earthquake occurs, these roads may be damaged along with other structures, hampering evacuation, rescue and repair activities in the stricken area. Past cases show, however, that where the function of the road was maintained the disaster was kept at a minimum.

Bridges are important structures to cross over obstacles; but they are liable to be affected by earthquakes. Such damage as the fall of the superstructure results in the loss of the function of the road and is difficult to repair. If the damage is not as severe as the fall of the supersturcture, temporary repairs may be made so that the bridge may be open to emergency traffic; it may be used semi-permanently, depending on the repair method.

There are 33,000 bridges on the national highways and prefectural roads with a total length of 22,000 km, and they vary in age, type, specifications, materials, etc. In order to ensure a certain level of safety for these bridges against earthquakes, it is necessary to devise appropriate methods of repair and retrofit and equipment.

Though complete regulations cannot be provided under the present circumstances, some measures have been taken for several important bridges and a repair manual has been prepared. This paper is intended to introduce a part of the manual together with some cases of repairs actually carried out.

II. Damage to Highway Bridges by the Past Earthquakes

The damage to the bridge was often extensive when the magnitude of the earthquake was relatively large. Let us review below the damage to the highway bridges caused by such big earthquakes as to have influenced the earthquake-resistant design for highway bridges (see Fig. 1).

Table 1 shows the number of bridges damaged by the Great Kanto Earthquake of 1923 (M = 7.9); Tables 2 and 3 give the breakdown of the damage in Tokyo and Yokohama. The damage caused by ground vibration was predominant in Yokohama as the city was close to the epicenter; but it decreased with distance from the epicenter.

The Fukui Earthquake (M = 7.3) of 1948 was a typical short-distance earthquake. The number of damaged bridges is given by Table 4. It is not possible to give the damage ratio here as the total number of bridges in the area at that time is not available.

The Niigata Earthqauke (M = 7.5) of 1964 damaged many structures due to large scale liquifaction on the saturated alluvium sandy ground. It was characteristic of the earthquake that the damage was mostly related to the problem of stability: settlement, tilting and sliding. The extent of the damage is shown by Table 5. Table 6 is intended to show the relationship between the damage and types of superstructure and of foundation with respect to the bridges within the 60 km radius of the city. Tables 7, 8 and 9 give the breakdown of the damage.

The Tokachi-oki Earthquake (M = 7.9) of 1968 causes extensive damage to roads, mainly to earth banks; but the damage to bridges was relatively small. Tbale 10 shows the extent of damage to bridges; Table 11 gives the breakdown of the damage.

The Miyagiken-oki Earthquake (M = 7.4) of 1978 caused damage mainly to structures. The damage was characterized by the fact that while many of those structures with their foundations on comparatively good bearing strata were damaged, the damage relating to stability, e.g., overturning and sliding, was smalo. Fairly extensive damage was caused to bridges, as shown by Table 12. Table 13, Figs. 3 and 4 give the breakdown of the damage into superstructure, support and substructure.

Table 14 gives the number of brdiges which suffered severe damage such as the fall of the superstructure in the past.

III. Methods of Repair and Retrofit for Highway Bridges

There is no prescribed method of repairing bridges damaged by earthquakes and of retrofitting existing bridges against earthquakes. But the Manual of Repair Work for Highway Bridges, published by the Japan Road Association in 1979, serves as a very useful guide.

As the damage to bridges takes various forms, repairs are usually carried out on a case by case basis. The procedure of repairing may be explained with a flwo chart shown as Fig. 5.

The procedure begins with inspection of the bridge concerned. It is necessary to inspect the structural dimensions, age, specifications, etc., as well as the extent of the damage. In this case it will be convenient if a check list is prepared prior to inspection.

Upon discovering a damaged secion, the extent and the form of the damage are to be ascertained in detail, e.g., failure, deformation, tilting, etc., in the light of the volume of emergency traffic and of future traffic. Before deciding as to whether the bridge can sustain the load of emergency traffic immediately after the disaster, it is necessary to ascertain if the bridge can be open to traffic with or without repairs or if it should be closed.

After order has been restored in the stricken area, the method of reconstruction will be selected. Depending on the durability, repair cost, future plan, etc. of the remaining structure, construction of a new bridge may be required.

After the method of retrofit for future earthquakes as well as that of repair has been selected, works at the site may commence. Effects of repairs and retrofit are to be examined upon completion of the works.

Among the existing bridges, there are many which were constructed to old specifications. The problem with roads is that the closing at one point of a route often leads to the loss of the function of the entire route.

Therefore, even in the case of an old bridge, it is necessary to take such measures as to ensure a certain level of safety against earthquakes corresponding to that of a new bridge. Past experience shows that unless the superstructure falls it is possible to make temporary repairs so that emergency traffic may not be obstructed. Accordingly, it was decided to take two of the three measures given below for all bridges located on trunk routes and they are now in progress.

(1) Minimum length of overlapping of girder and coping at support.

The minimum length S in Fig. 6 may be prescribed as:

$$
\begin{aligned}
S &= 70 + 0.5\ell \quad (\ell \leqq 100) \\
  &= 80 + 0.4\ell \quad (\ell > 100)
\end{aligned}
$$

where S: minimum length (cm)
ℓ: span length (m)

(2) Devices for preventing dislodgement

The types of devices are shown in Fig. 7.

(3) Connecting divices for neighboring superstructures

The types of divices are shown in Fig. 8.

IV. Examples of Repairs and Retrofit

There have been many examples of repairing bridges damaged by earthquakes. But the old methods are not applicable to the present bridges. So some examples and methods of repairing damages caused by recent earthquakes are given below.

Those parts of bridges damaged by earthquakes can be classified into superstructure, support and substructure as mentioned before.

As the damage to the superstructure is concentrated in expansion joint, handrail, buckling of sway bracing, it is easy to replace them. The buckling of web plate can be reinforced with stiffener. As a special case, repairing works on the side span of the Bandai Bridge in Niigata Earthquake are illustrated in Fig. 9.

The main damage to the support is shown in Fig. 2. In many cases, they are repaired by jacking up the superstructure as shown in Fig. 10. If jacking is difficult, another temporary support, serving to secure the length of overlupping of girder and coping, shall be prepared as shown in Fig. 11.

The damage to the substructure can be divided into two types. One is related to stability, e.g., as settlement, tilting, sliding and so on. Another concerns safety of structures, e.g., failure, breakage, crack and so on.

For the former case, there are several methods such as underpinning, filling up, etc. However in most cases the cost is so high that reconstruction is advantageous.

For the latter, such method as wrapping with reinforced concrete as shown in Fig. 12, is common. Besides, there are many cases where partial repairs are sufficient to keep the bridges open. But in the case of the structural damage, it is dangerous even for emergency traffic, some emergency measures are taken as in Fig. 13. In Fig. 14 typical examples of the cases requiring either emergency measures or permanent repairs.

Recently, projects to give retrofit to old bridges against earthquakes are in progress in Japan. Most of them are works to expand the length of overlupping on the support confirming to current specifications, to install devices to prevent dislodgement and to attach the connecting divices between neighboring beams.

The latter two are relatively easy works as shown in Figs. 7 and 8. For the former some methods as Figs. 15 and 16 are used.

Sometimes the retrofitting of the substructure of old bridges against earthquake is adopted because they have not been designed according to earthquake resistant regulations. In Fig. 17 one example of the retrofitting of an abutment is explained. The increment of pile is planned in a curious shape depending on a narrow space around the existing bridge. Fig. 18 is one example of the retrofitting of a pier.

In Japan, the foundations are often exposed because of the lowered riverbed due to the heavy demand for gravel. Accordingly, these foundations have become dangerous in earthquakes because of decreased lateral ground resistance. As a countermeasure, one example of the stabilization of riverbed is shown in Fig. 19.

A cast in site diaphragms wall method used in Fig. 19 and explained in Fig. 20 is recently becoming one of the most effective methods for retrofitting.

V. Discussion

So far the authors have introduced the current methods of repairing and retrofitting against earthqaukes in Japan. But they are not methodical and not systematical. So we must apply them on a case by case basis. However, it is very difficult to evaluate the damage and to decide on the repair method during the confusion following an earthquake. Therefore, an appropriate guide is desirable.

On the other hand, the planning of retrofitting for old bridges to give some resistance against earthquakes is also difficult because in many cases their figures and records of calculation have been not kept. Therefore some regulated methods are required.

In this connection, the subjects to be studied in future may be listed as below.

(Repairing)

Method for survey and inspection

- Composition of a check list for inspection
- Evaluation method of load carrying
- Capacity for damaged structure
- Manual for the selection of repair method
- Inspection method for repaired structure

(Retrofitting)

Method for survey

- Evaluation method for earthquake resistance of existing structures
- Estimation of durability for existing structures
- Manual for the selection of retrofitting method
- Evaluation method for effects of retrofitting

Table 1 Total number of bridges damaged in the Kanto earthquake of 1923[1)]

Prefectures or Cities	Total Number of Bridges Surveyed	Number of Bridges Damaged due to Vibration and/or Fires	Percentages of Damage	Remarks
Tokyo	3,338	230	6.9%	Except city of Tokyo
City of Tokyo	675	358	53.0%	
Kanagawa	1,253	893	71.3%	Except city of Yokohama
City of Yokohama	108	91	84.2%	
Shizuoka	358	100	27.9%	Inside the affected area (Numazu or Northern area)
Saitama	1,313	27	2.1%	
Yamanashi	245	21	8.6%	Only wooden bridges suffered inside the affected area
Chiba	690	65	9.4%	
Total	7,980	1,785	22.4%	

Table 2 Damage characteristics in the City of Tokyo[1)]

Type of Bridges	Total Number of Bridges Surveyed	Number of Bridges Damaged and Percentages		
		Caused by Vibration	Caused by Fires	Total
Wooden	420	6(1.4%)	276(65.7%)	282(67.1%)
Steel	60	6(10.0%)	49(81.7%)	55(91.7%)
Masonry	144	2(1.4%)	5(3.5%)	7(4.9%)
Plain concrete	4	4(100%)	0(0%)	4(100%)
Reinforced concrete	47	0(0%)	10(21.3%)	10(21.3%)
Total	675	18(2.7%)	340(50.3%)	358(53.0%)

Table 3 Damage characteristics in the City of Yokohama[1)]

Type of Bridges	Total Number of Bridges Surveyed	Number of Bridges Damaged and Percentages			
		Caused by			Total
		Vibration + Fires	Vibration Only	Fires Only	
Wooden	75	26(34.6%)	25(33.4%)	8(10.7%)	59(78.7%)
Steel	31	11(35.5%)	16(51.6%)	3(9.7%)	30(96.8%)
Reinforced concrete	2	0(0%)	2(100%)	0(0%)	2(100%)
Total	108	37(35.2%)	43(39.8%)	11(10.2%)	91(84.2%)

Table 4 Statistics on damage to highway bridges due to the Fukui earthquake of 1948[1)]

Prefectures	Bridge Damage		Highway Damage Except Bridges	
	Number of Bridges	Repairing Cost	Number of Sites	Repairing Cost
		Thousand Yen		Thousand Yen
Fukui	180	189,869	475	205,945
Ishikawa	63	17,782	155	41,463
Total	243	207,651	630	247,408

(Note) Amount of loss was evaluated at the value at the time of the earthquake.

Table 5 Statistics on damage to highway bridges (except wooden bridges) due to the Niigata earthquake of 1964[1)]

Prefectures	Number of Damaged Bridges	Number of Severely Damaged Bridges	Number of Fallen Bridges	Approximate Epicentral Distance
Akita	7	0	0	140 - 160 km
Fukushima	5	0	0	120 - 150 km
Niigata	74	8	3	30 - 100 km
Yamagata	12	0	0	60 - 100 km
Total	98	8	3	

Table 6 Damage percentages of individual portions of highway bridges within 60 km from the center of Niigata City[1)]

Structural Classification		Type of Structures	Number of Structures Surveyed	Damaged Structures: Number of Structures	Damaged Structures: Percentages
Super-structures		Steel Girders	168 spans	19 spans	11.3 %
		Reinforced Concrete Girders	222 spans	33 spans	14.9 %
		Prestressed Concrete Girders	132 spans	11 spans	8.3 %
		Wooden Girders	8 spans	8 spans	100 %
		Total	530 spans	71 spans	13.4 %
Substructures	Abutments	With Spread Footings	24	4	16.7 %
		With Pile Foundations	99	19	19.2 %
		With Caisson Foundations	29	7	24.0 %
		Sub-Total	152	30	19.7 %
	Piers	With Spread Footings	40	0	0 %
		With Pile Foundations	214	21	9.8 %
		With Caisson Foundations	180	15	8.3 %
		Sub-Total	444	36	8.1 %
		Total	596	66	11.1 %

Table 7 Type and number of damages of superstructure in Niigata earthquake[4)]

Part \ Damage	Peeling	Crack	Yield	Failure	Buckling	Deforma-tion	Crash	Shear	Dis-placement	Slip out	Break-age	Fall
1. Pavement	14	15										
2. Handrail	7	14		6	1	2	1		4	2	4	
3. Expansion Joint		1							9			
4. Slab	2	5					2		2			
5. Floor System		1										
6. Sway and Lateral					1	2			2			
7. Main Structure	3	12	2				1		11	1		6
8. Sole Plate								4	2			
9. Bearing								5	2			
10. Mortar under Bearing	10	10					1		2			
11. Embedded Bolt									2	5		

Table 8 Type and number of damages of substructure in Niigata earthquake[4)]

Part \ Damage	Peeling	Crack	Yield	Failure	Buckling	Deforma-tion	Dis-placement	Break-age	Tilting	Slid-ing	Over-turning	Settle-ment
1. Support	1					2		5	26	13	2	32
2. Coping	10	16		4		1						
3. Body		9		1	3	4		5	26	13	2	32
4. Footing												
5. Foundation												

* Tilting, sliding, overturning and settlement are to be considered in terms of the whole substructure.

Table 9 Type and number of damaged of approach road in Niigata earthquake[4)]

Part \ Damage	Peeling	Crack	Yield	Failure	Buckling	Deforma-tion	Dis-placement	Break-age	Tilting	Slid-ing	Over-turning	Settle-ment
6. Approach Portion		10		1			1	2		4		22
7. Bank		4					1			2		14
8. Retaining Wall	2	8				8	1		18	23		22
9. Wing	4	16				6	4		5	16	1	14

Table 10 Statistics on damage to highways and highway bridges due to the Tokachi-oki earthquake of 1968

Administrative Agency		Highways		Highway Bridges		Total		Remarks
		Number of Sites	Loss	Number of Bridges	Loss	Number	Loss	
Ministry of Construction		111	Thousand Yen 969,604	-	-	111	Thousand Yen 969,604	Mostly to National Highways in Aomori Prefecture
Prefectural Governments	Hokkaido	63	319,359	30	Thousand Yen 206,176	93	525,535	
	Aomori	818	1,167,060	64	187,770	882	1,354,830	
	Iwate	41	34,178	3	19,000	44	53,178	
	Miyagi	-	-	3	5,100	3	5,100	
	Akita	4	4,600	1	3,000	5	7,600	
Total		1,037	2,494,801	101	421,046	1,138	2,915,847	

Note) Amounts of loss were estimated at the value as of June, 1968.

Table 11 Type and number of damages in Tokachi-oki earthquake of 1968[5)]

Damage		Failure	Movement	Deformation	Settlement	Inclination	Buckling	Crack	Fall of covering of RC
Damages to Super-structures	Main girder		7	1			2	2	1
	Pavement							2	
	Handrail	2	1	1					
	Shoe and its fixed mortar	4	7					4	3
Damages to Sub-structures	Abutment	6	6		4			14	4
	Pier				3	1		3	3
	Foundation								
	Ground				1			1	
Damages to Approaches	Embankment in contact with abutment	2	3		2			2	
	Embankment	1	4		3			1	
	Retaining wall and revetment	3	11	1				11	

Table 12 Whole bridges and damaged bridges in Miyagiken-oki earthquake[3)]

	Number of Whole Bridges	Number of Damaged Bridges			
		Super-structure	Bearing	Sub-structure	Total Number of Damaged Bridge
Under the Government	55	2	21	35	42
Under the Governor of Miyagi	400	13	43	72	95
Under the Governor of Iwate	170	6	18	8	20
Under the Governor of Fukushima	4	1	2	0	2
Under the Japan Highway Corp.	269	0	40	18	47
Total	898	22	124	133	203

Table 13 Damages of superstructure in Miyagiken-oki earthquake[3)]

Steel

Name of Bridge	Type	In augu-lation	Damage
1 Kinnoh	Cantilever Girder (Gerber)	1956	Fall of Suspended Girder
2 Yanaizu	Continuous Truss	1975	Buckling of Web Plate on Fixed Support
3 Maiya	Cantilever Truss (Gerber)	1928	Breakage of Upper Chord of Truss
4 Date	Continuous Truss	1965	Buckling of Web Plate on Fixed Support
5 Sin Eai	Continuous Girder	1974	Buckling of Sway Bracing on Support
6 Fukuda	Cantilever Girder (Gerber)	1955	Buckling of Sway and Lateral Bracings Near Support, Failure of Rivets.
7 Fuji	Continuous Girder	1971	Buckling of Sway Bracing on Support
8 Sakunose	Langer Girder	1962	Deformation of Portal and Lateral Bracing on Support
9 Iwaigawa	Continuous Girder	1977	Buckling of Sway Bracing on Support
10 Wagagawa	ditto	1969	ditto

Concrete

11 Yuriage	PS Beam	1972	Crack of Beam on Support
12 Dohdono, 10 others	RC Beam and RC Cantilever Beam	-	Crack of Beam on Support

Table 14 Number of bridges fallen and burned by earthquakes after Kanto earthquake

Name of Earthquake	Year	Fallen	Burned
Kanto	1923	8	9
Nankai	1946	1	0
Fukui	1948	7	0
Niigata	1964	3	0
Miyagiken-oki	1978	1	0
Total		20	9

No	Date	Name	M
1	Spet. 1, 1923	Kanto	7.9
2	Dec. 21, 1946	Nankal	8.1
3	Jun. 28, 1948	Fukui	7.3
4	Dec. 26, 1949	Imaichi	6.4
5	Mar. 4, 1952	Tokachi-oki	8.1
6	Apr. 30, 1962	Northern Miyagi	6.5
7	Jun. 16, 1964	Niigata	7.5
8	Feb. 21, 1968	Ebino	6.1
9	May 16, 1968	Tokachi-oki	7.9
10	Jun. 12, 1978	Miyagiken-oki	7.4

(*) Magnitudes are on the Richter scale, after Rika Nenpyo (Annual Report of Science) (1971).

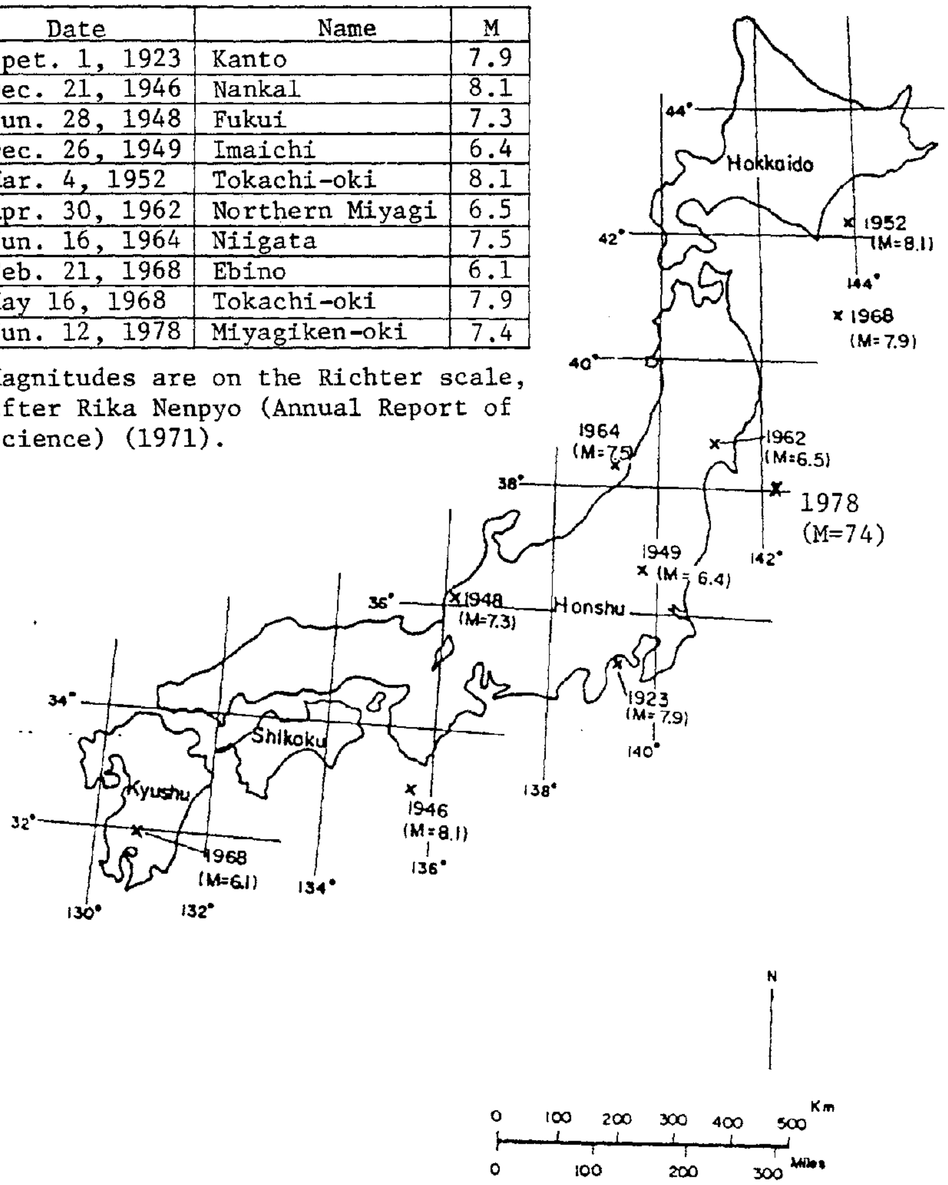


Fig. 1 Epicenters of ten earthquakes which caused comparatively severe damage to modern engineering structures in Japan[1])

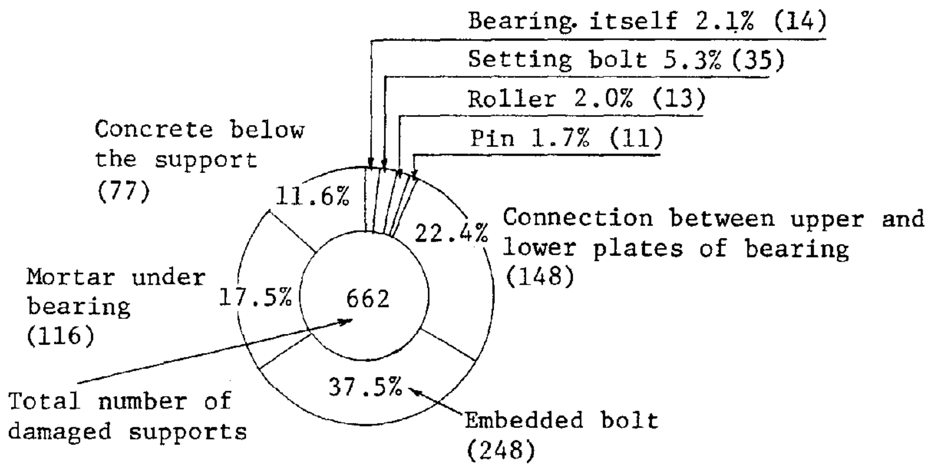


Fig. 2 Breakdown of damaged supports[3)]

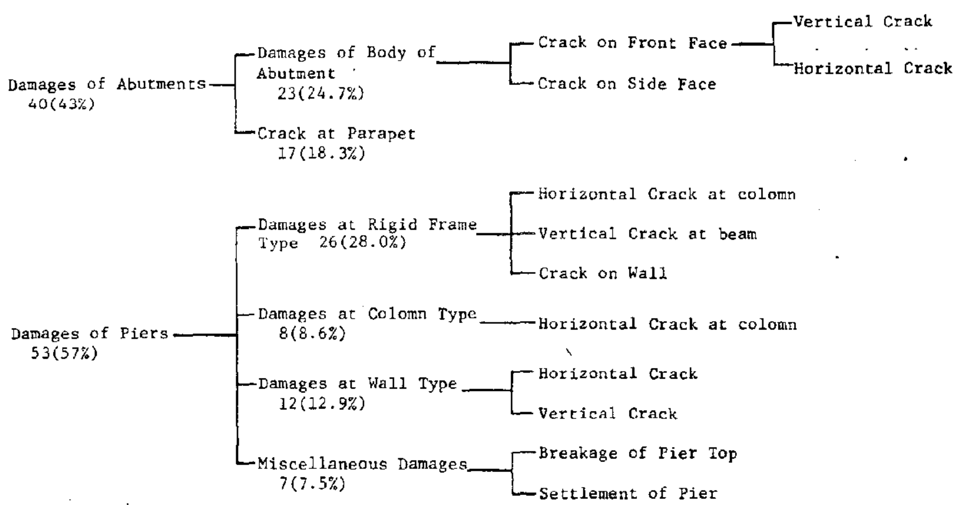
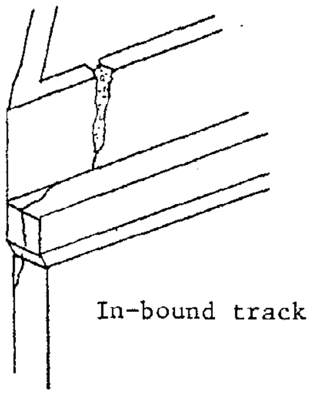
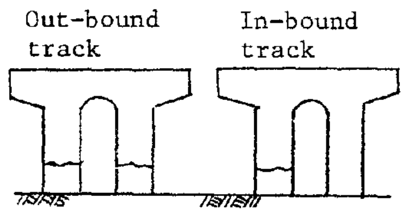


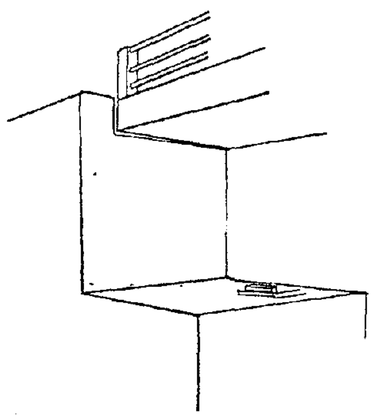
Fig. 3 Classification of damages of substructures[3)]



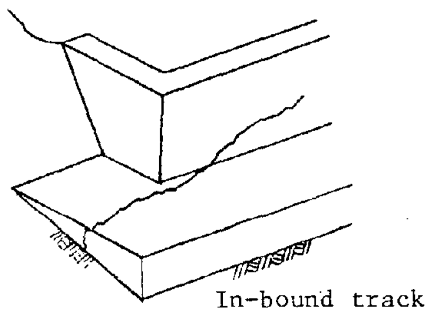
1. Nagamachi Overpass (A_1)



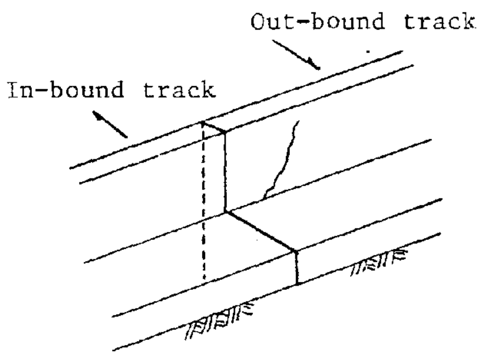
2. Umedagawa Br. (P_1)



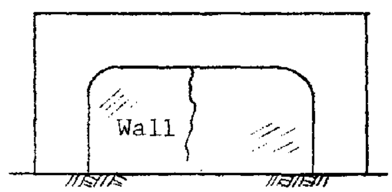
3. Umedagawa Br. (A_2)



4. Izumi Br. (A_1)

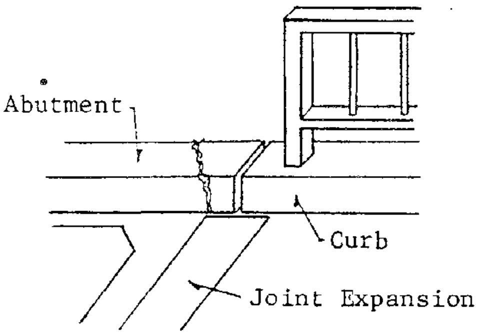


5. Izumi Br. (A_1)

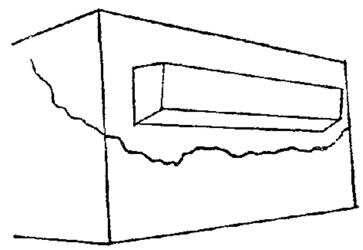


6. Tadagawa Br. (P_1)

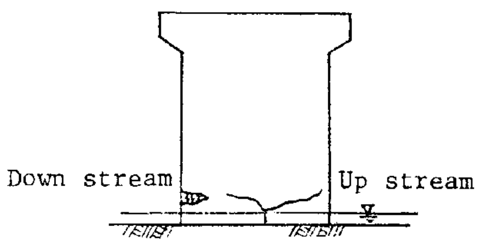
Fig. 4 Damages of abutments and piers in Miyagiken-oki earthquake (1/11)



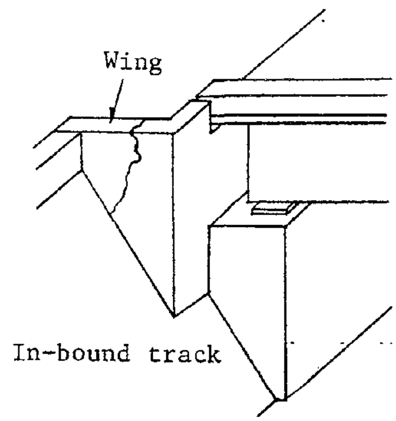
7. Tadagawa Pedestrian Br. (A_2)



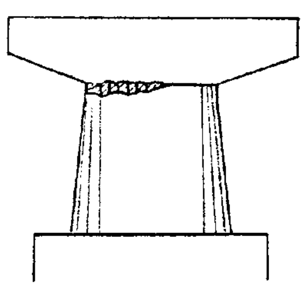
8. Sin Eai Br. (A_1)



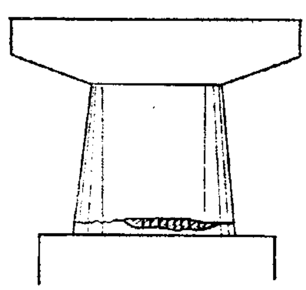
9. Sin Eai Br. (P_3)



10. Arase Br. (A_1)

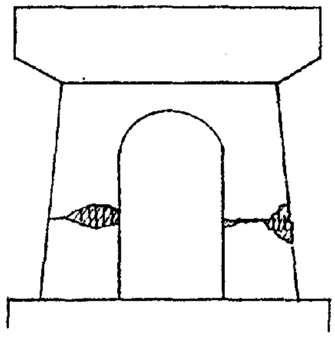


11. Sendai Br. ($P_5 \sim P_8$)

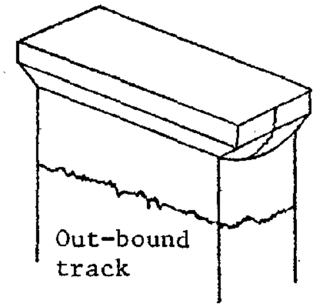


12. Sendai Br. ($P_1 \sim P_4$, P_7)

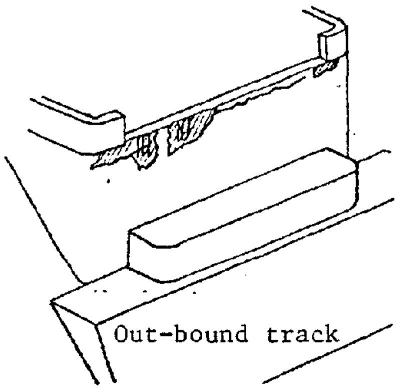
Fig. 4 (2/11)



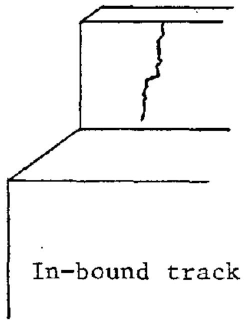
13. Abukuma Br. ($P_2 \sim P_9$)



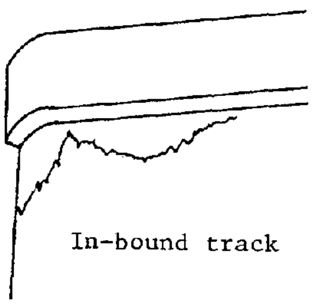
14. Fukuda Br. (P_2)



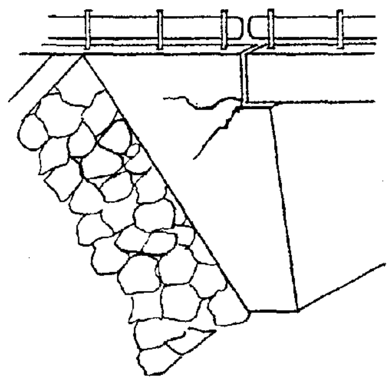
15. Fukuda Br. (A_2)



16. Fukuda Br. (A)

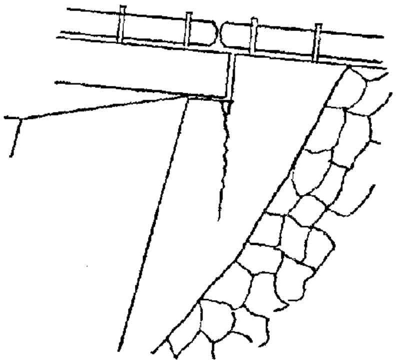


17. Fukuda (P_3)

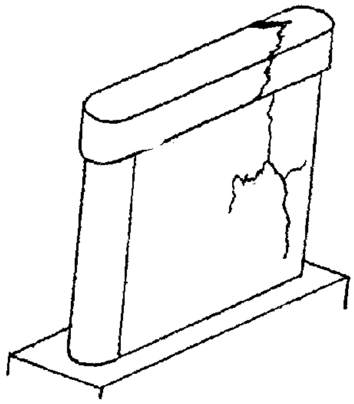


18. Hebita Overpass (A_1)

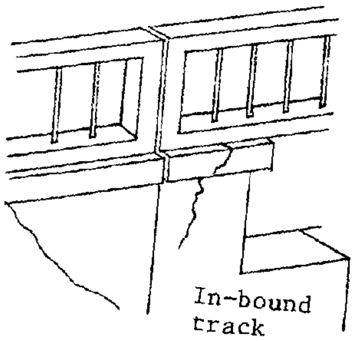
Fig. 4 (3/11)



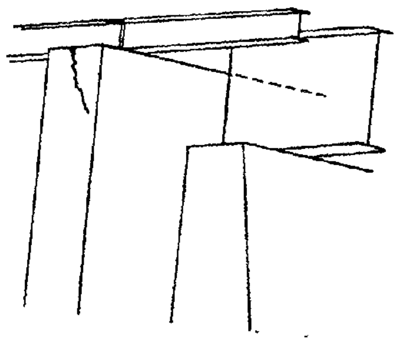
19. Hebita Overpass (A_2)



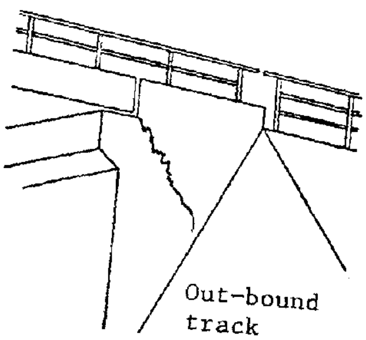
20. Tenno Br. (P_1)



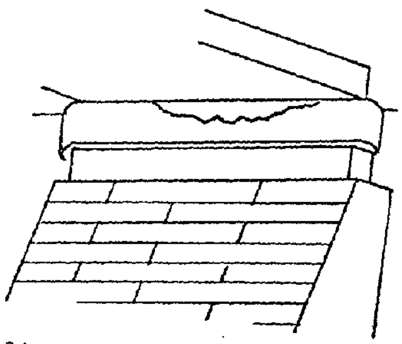
21. Iwadeyama Br. (A_1)



22. Kawawatari Pedestrian Br. (A_2)

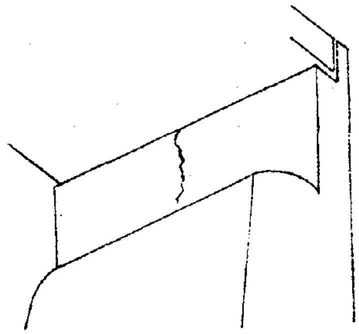


23. Ayashi Overpass (A_1)

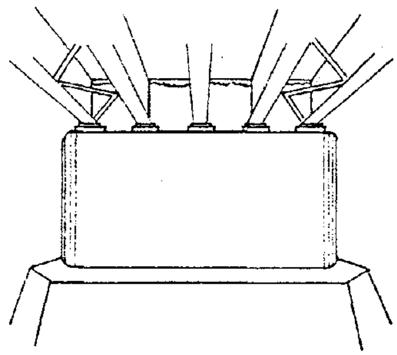


24. Takakuragawa Br. (A_2)

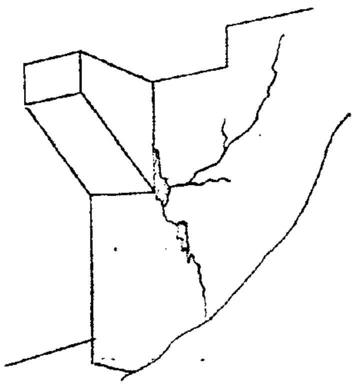
Fig. 4 (4/11)



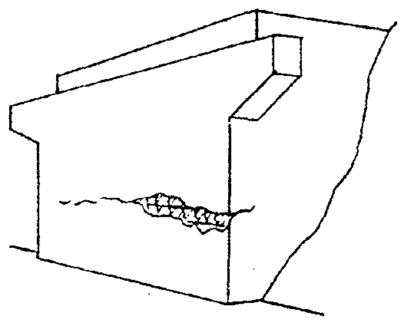
25. Natori II Br. (A_R)



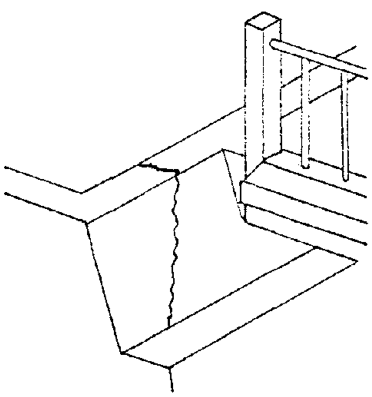
26. Ochiai Br. (A_L)



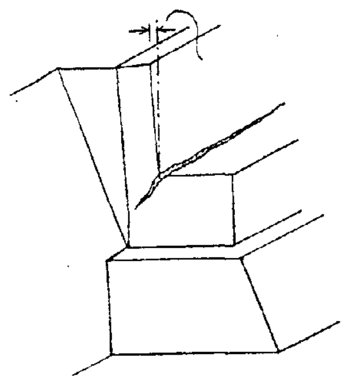
27. Nanakitada Br. (A_L,A_R)



28. Nanakitada Br. (A_R)

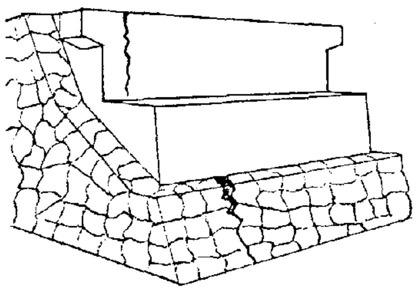


29. Sin Abumikawa Br. (A_L,A_R)

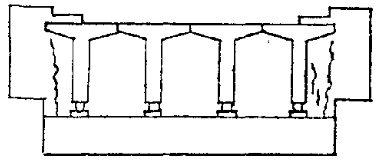


30. Yaotome Br. (A_L,A_R)

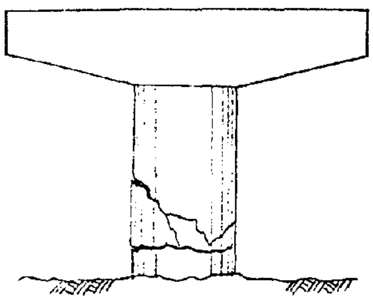
Fig. 4 (5/11)



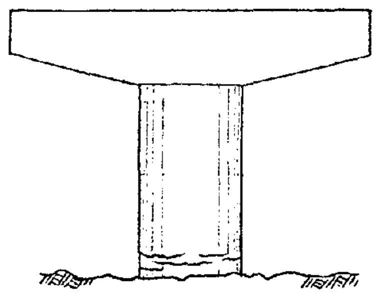
31. Shinainuma Br. (A_R)



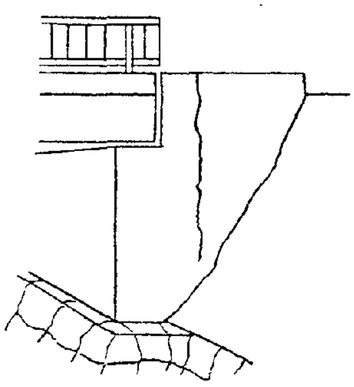
32. Yuriage Br. (A_L)



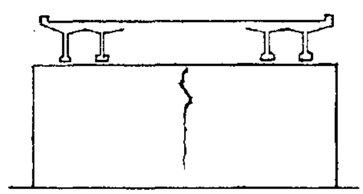
33. Yuriage Br. (P_1)



34. Yuriage Br. ($P_3 \sim P_9$)

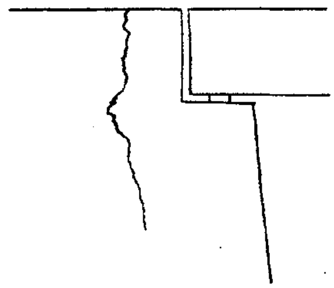


35. Takasago Br. (A_L)

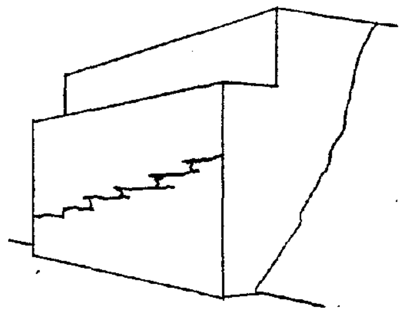


36. Nakanoshima Br. (A_R)

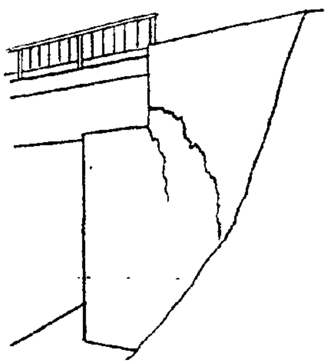
Fig. 4 (6/11)



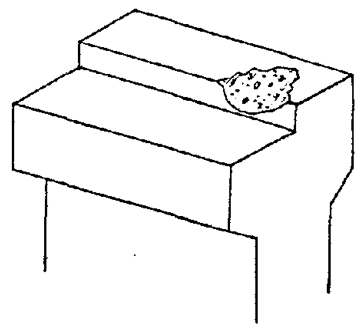
37. Ohshiro Br. (A_L)



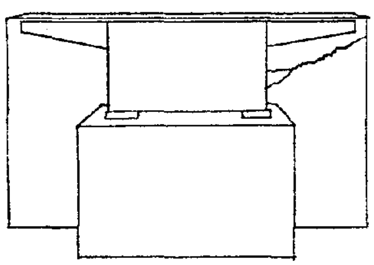
38. Shimo Tsuruta Br. (A_L)



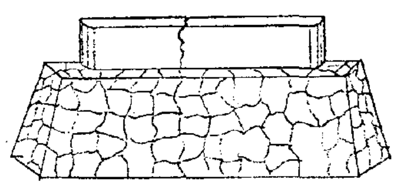
39. Shimo Tsuruta Br. (A_L)



40. Shimo Tsuruta Br. (P_1)

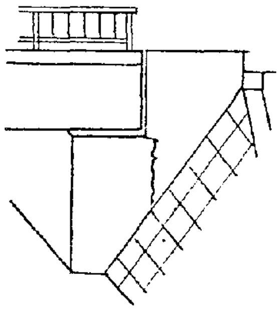


41. Kaihoku Br. (A_L)

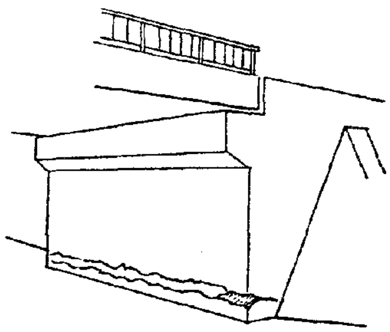


42. Tsujido Br. (A_L)

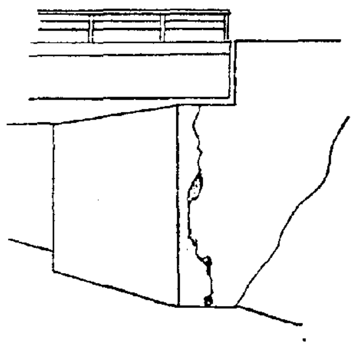
Fig. 4 (7/11)



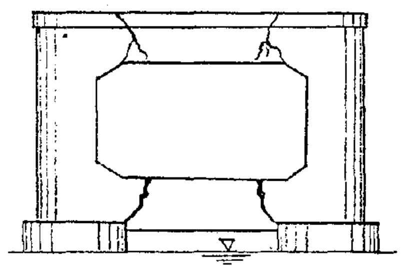
43. Ohwada Br. (A_L)



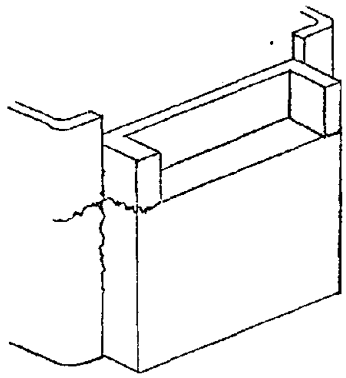
44. Tamachi Br. (A_L)



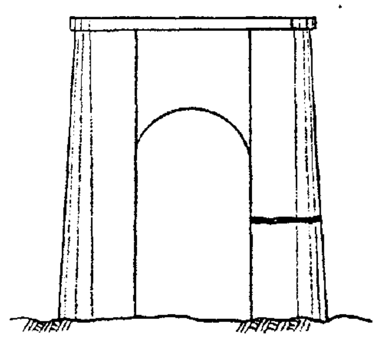
45. Takakawa Br. (A_L,A_R)



46. Eaigawa Br. ($P_1 \sim P_8$)

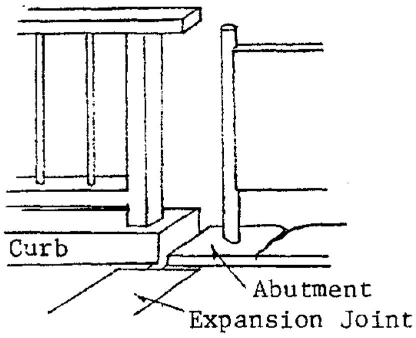


47. Eaigawa Br. (A_L)

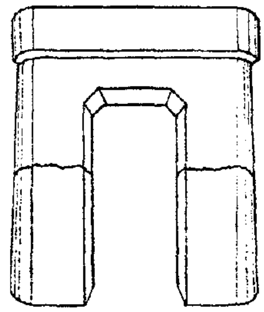


48. Noda Br. (P_3)

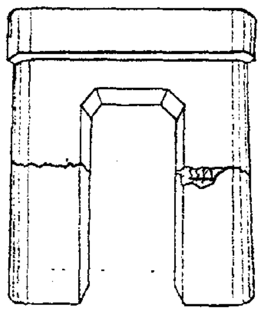
Fig. 4 (8/11)



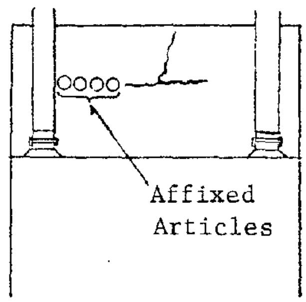
49. Shikitama Br. (A_R)



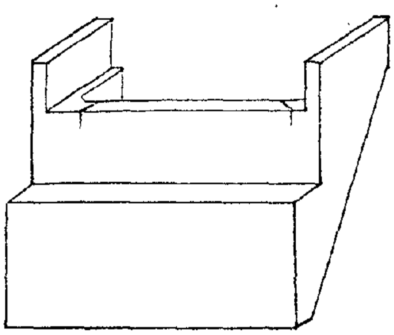
50. Shida Br. (P_3,P_6)



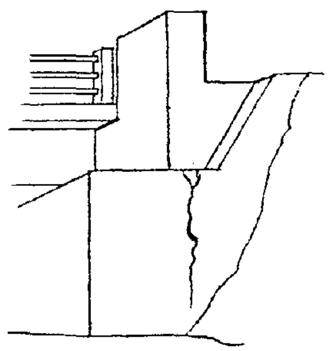
51. Shida Br. (P_4)



52. Sakuranome Br. (A_R)

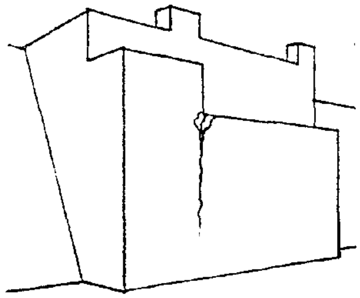


53. Fujiya Overpass (A)

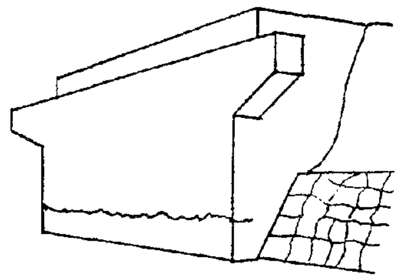


54. Daiji Br. (A_L)

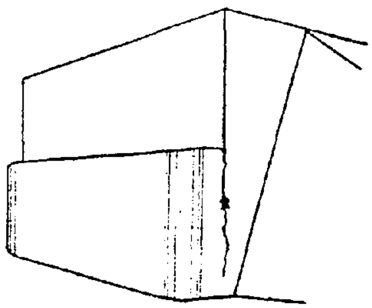
Fig. 4 (9/11)



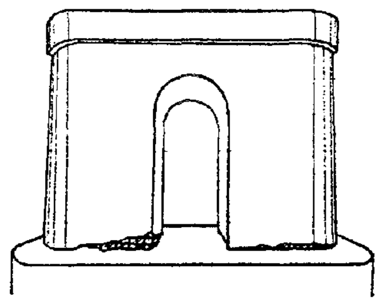
55. Daiji Br. (A_L)



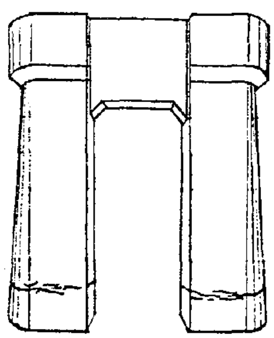
56. Wakayanagi Br. (A_L)



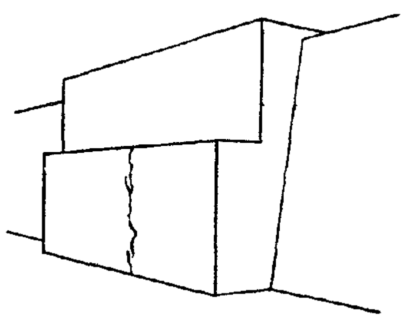
57. Shimoda Br. (A_R)



58. Tome Br. (P_{12})

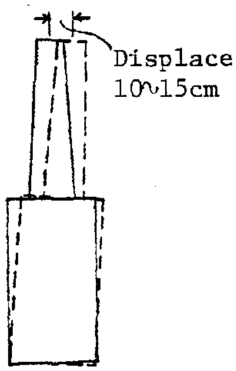


59. Kinnoh Br. (P_8)

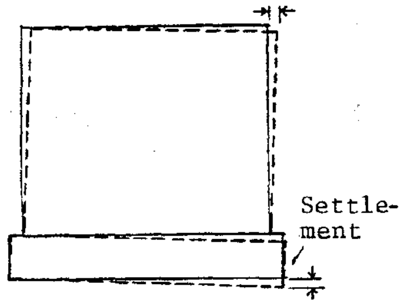


60. Namiita Br. (A_L)

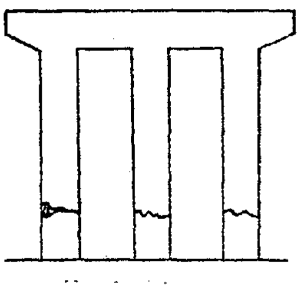
Fig. 4 (10/11)



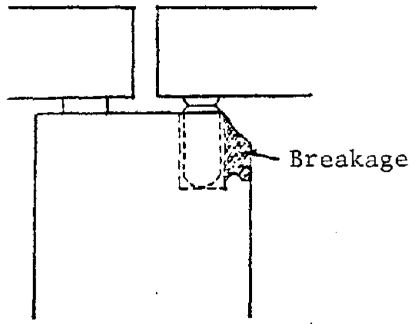
61. Kitakamigawa Br. (P_1)



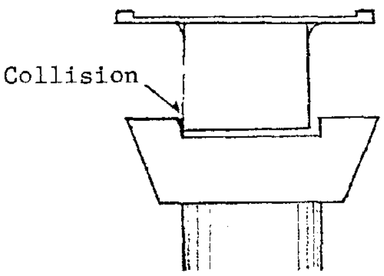
62. Ipponsugi Br. (P_1)



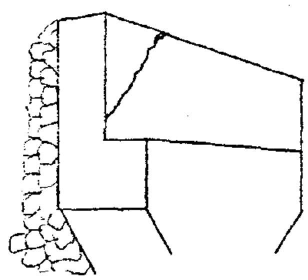
63. Konai Overpass (P)



64. Shinonome Br. (P_4,P_6)



65. Kunimi Br.



66. Sakunose Br. (A_R)

Fig. 4 (11/11)

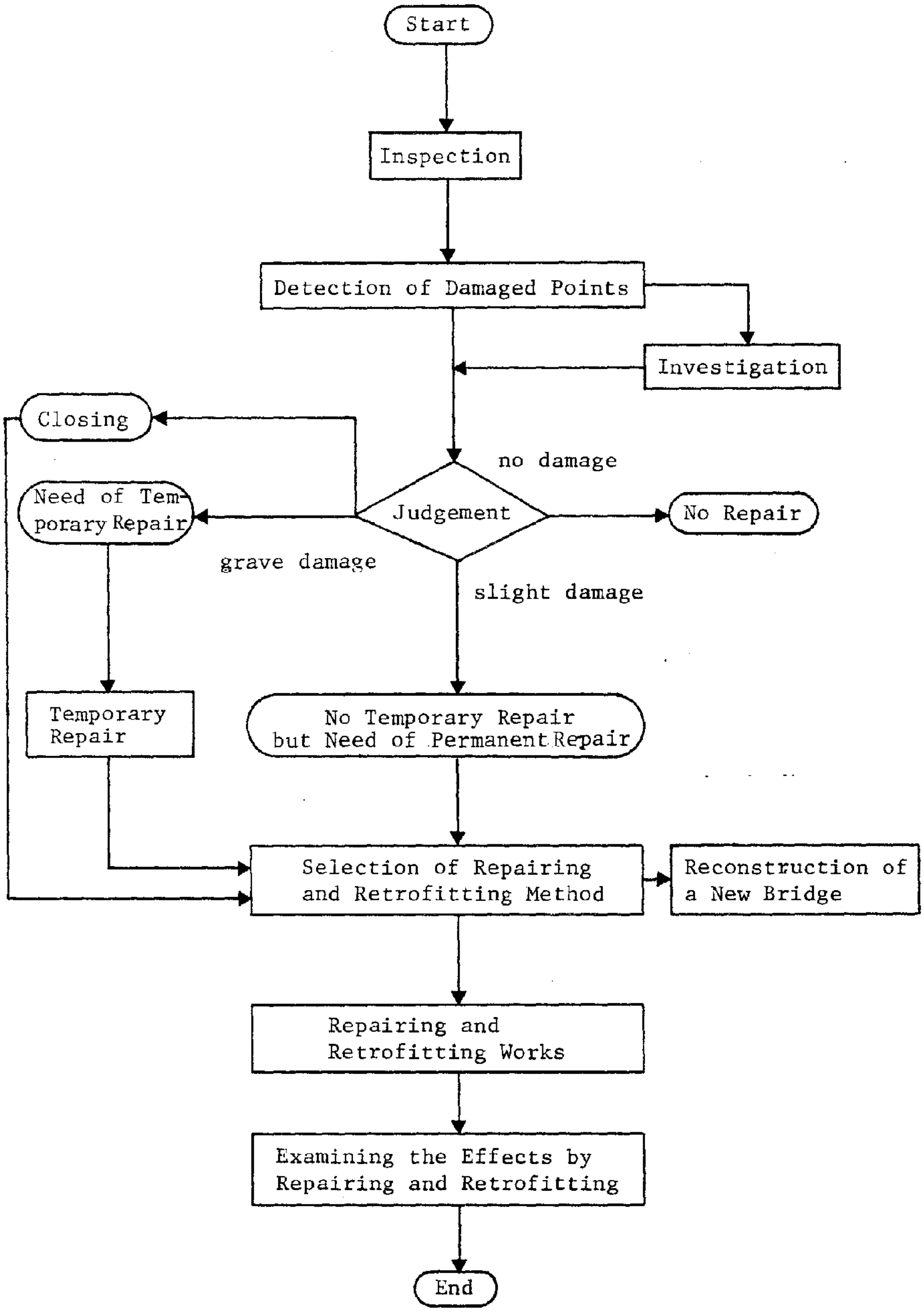


Fig. 5 Flow of repair and retrofit for bridge

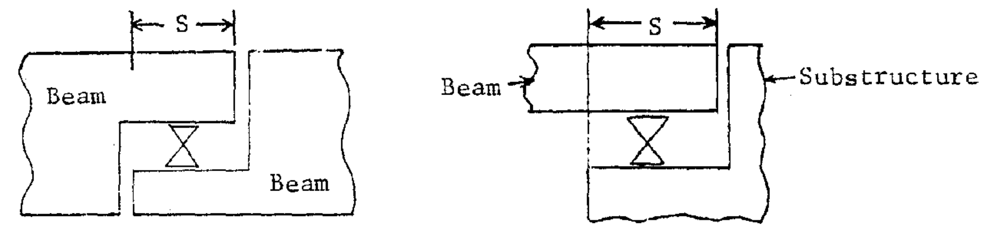


Fig. 6 Length of overlup on support

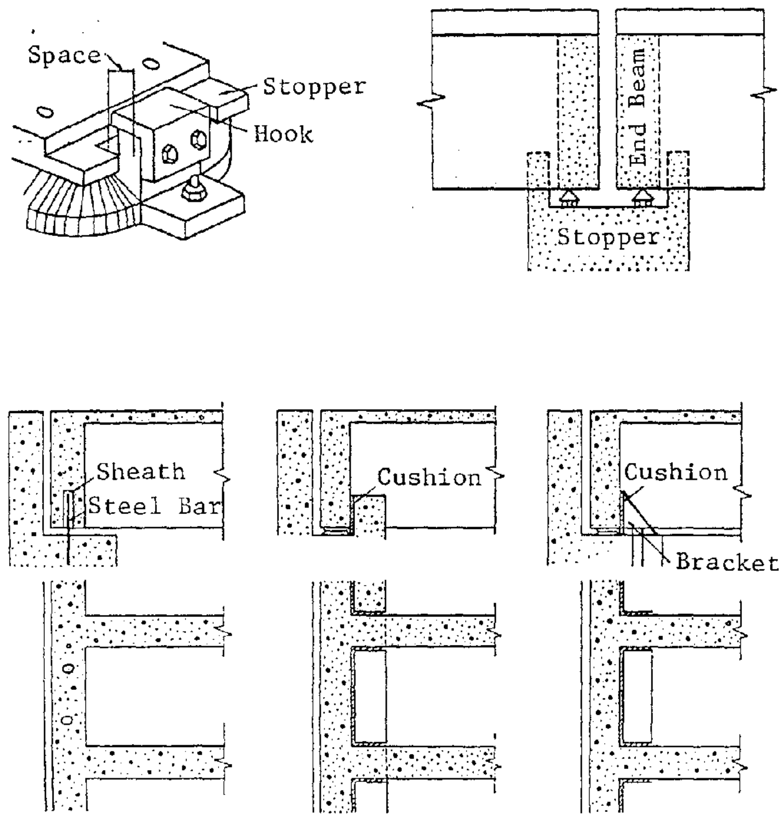
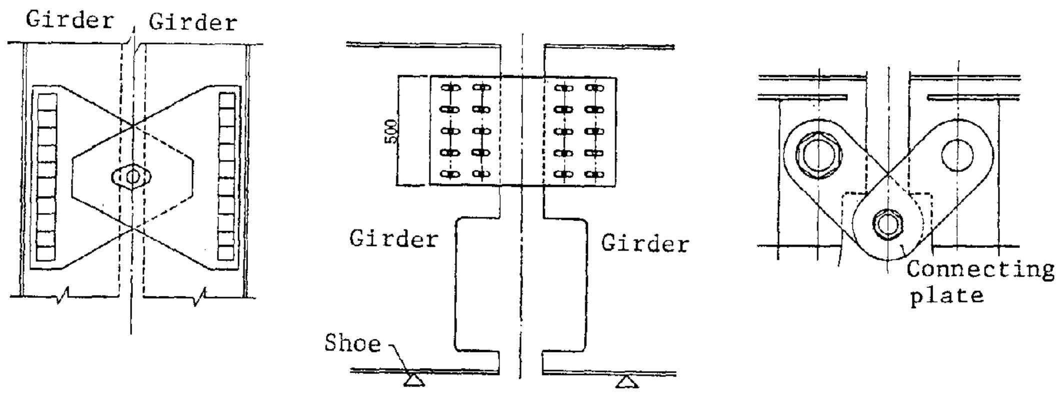
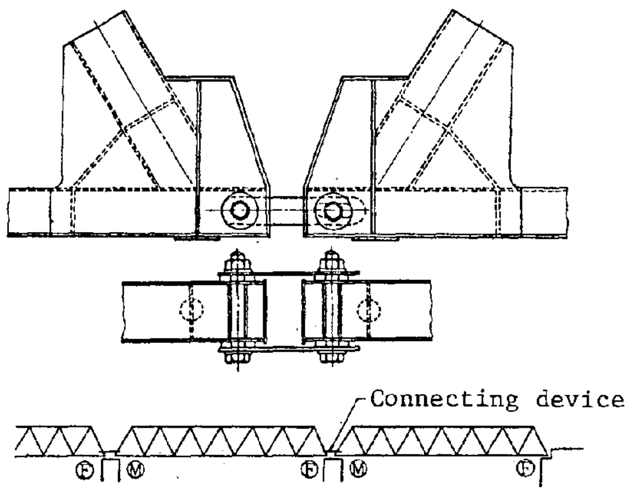


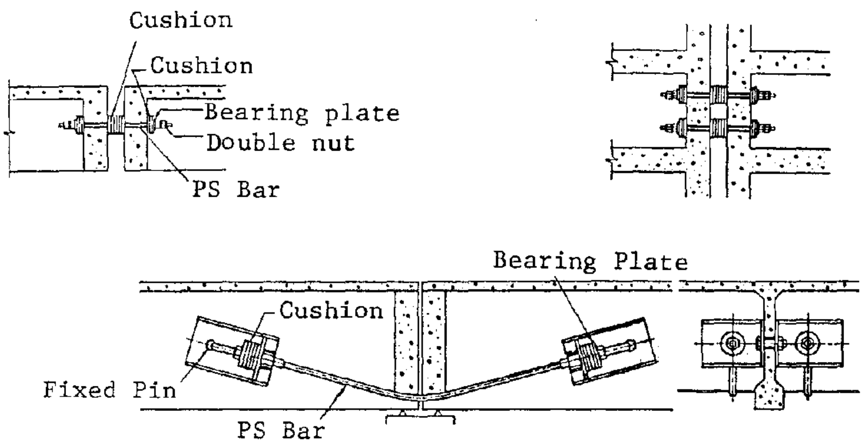
Fig. 7 Devices preventing dislodgement[2)]



For Steel Girder

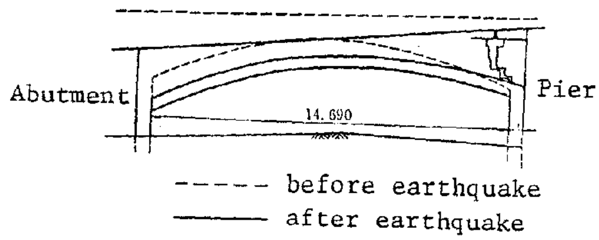


For Truss

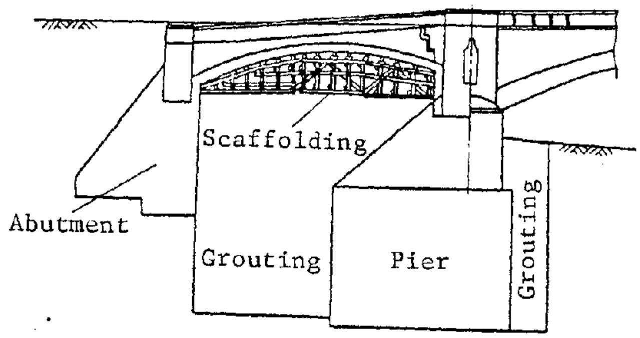


For PS Beam

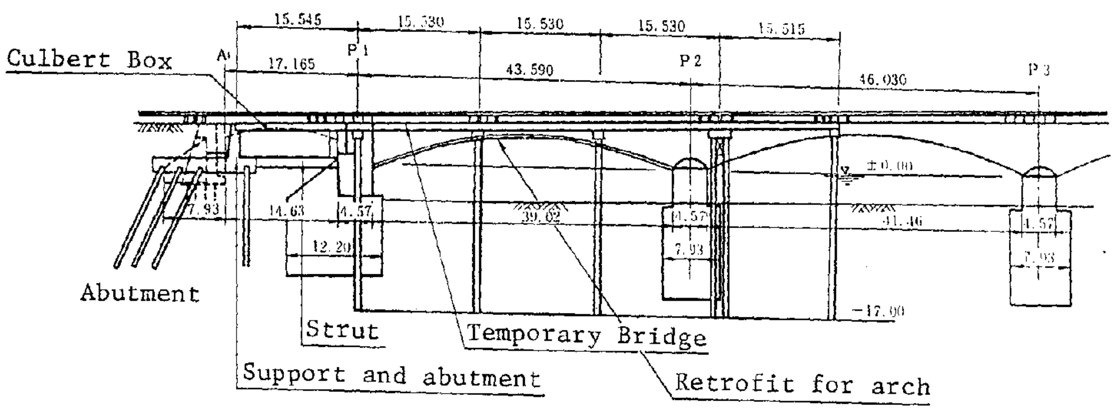
Fig. 8 Connecting devices[2)]



Damage at end span



Urgent Step



Real repairing & retrofitting

Fig. 9 Repairing for Bandai bridge in Niigata[8)]

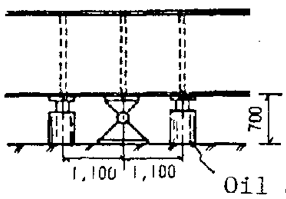


Fig. 10 Jacking up [6)]

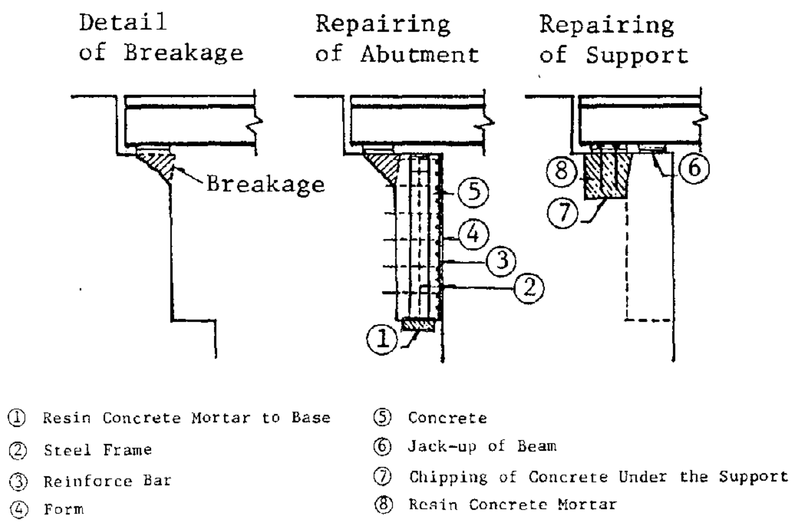


Fig. 11 A example of repairing and retrofitting of support [10)]

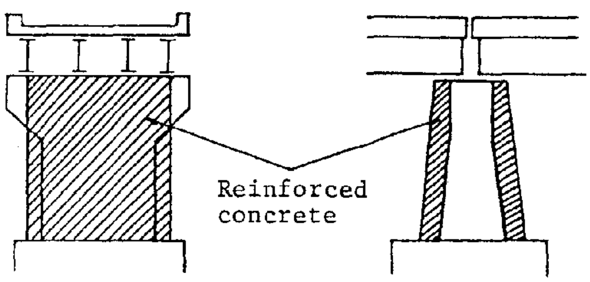


Fig. 12 Wrapping with Reinforced concrete [7)]

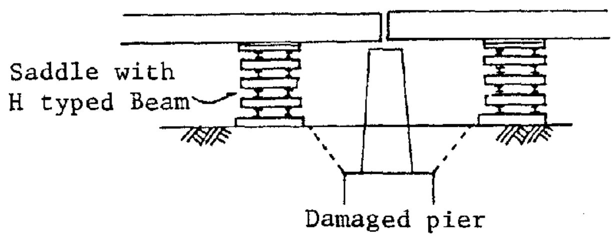


Fig. 13 Temporary Support[7)]

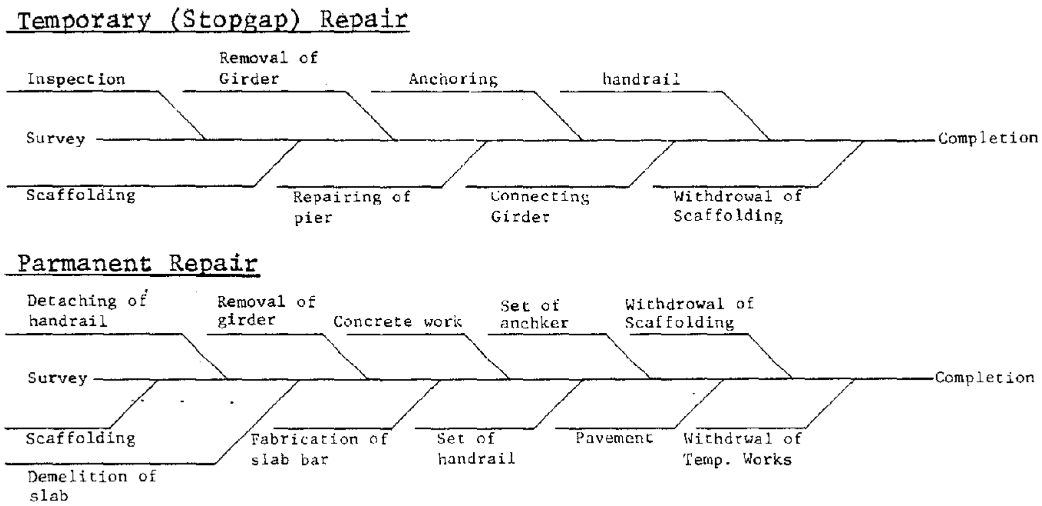


Fig. 14 One example of repairing procedure[6)]

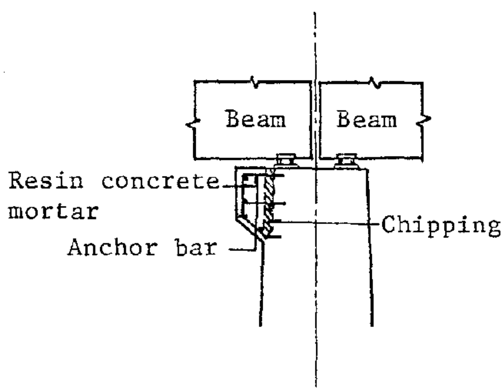


Fig. 15 Widening of coping[10)]

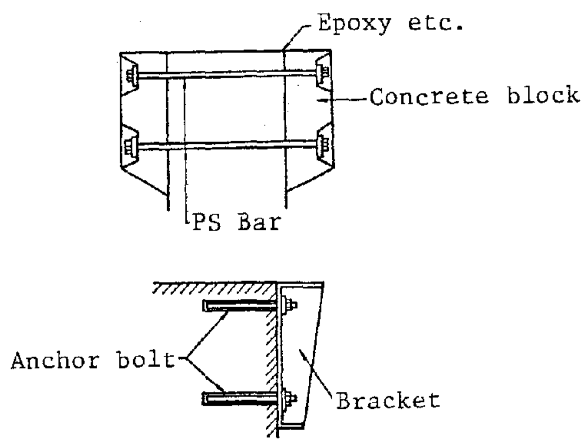
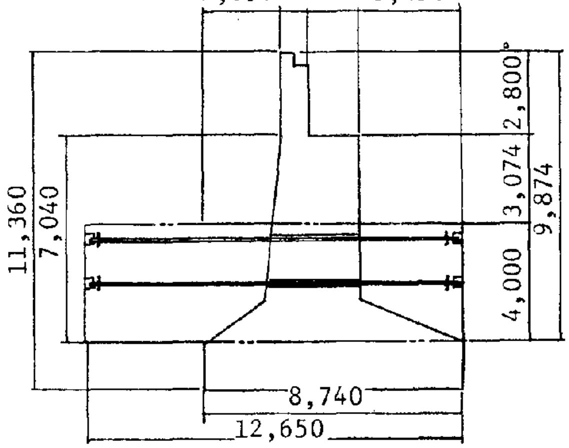
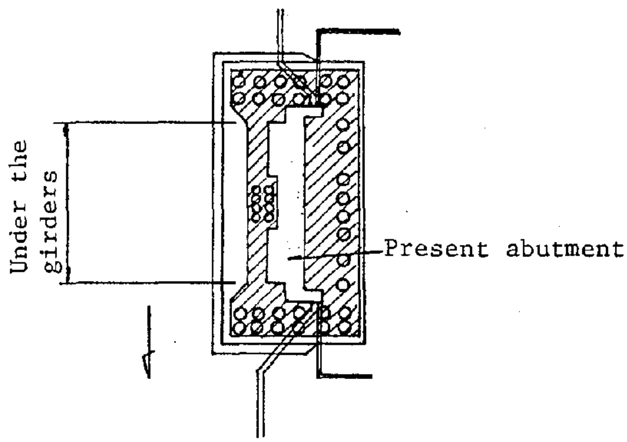


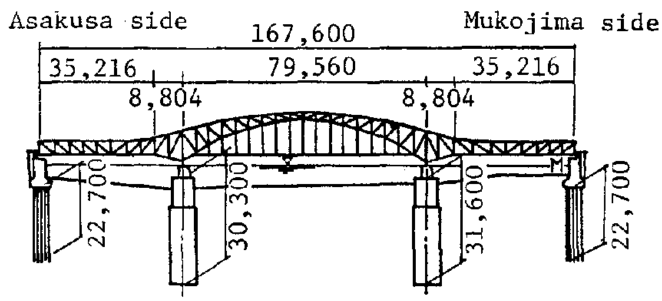
Fig. 16 Special method for widening of coping



Widening of footing



Increment of piles



Retrofit for abutments

Fig. 17 Retrofitting of abutment for earthquake[10)]

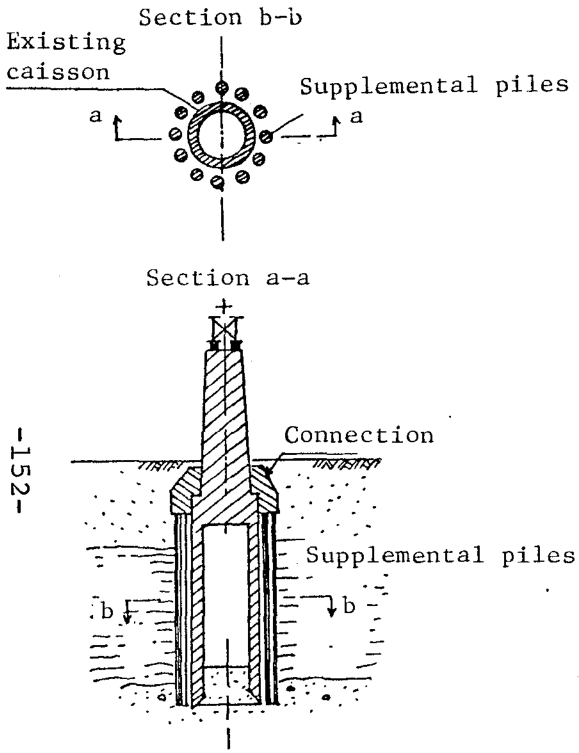


Fig. 18 Retrofit for insufficient bearing capacity[10)]

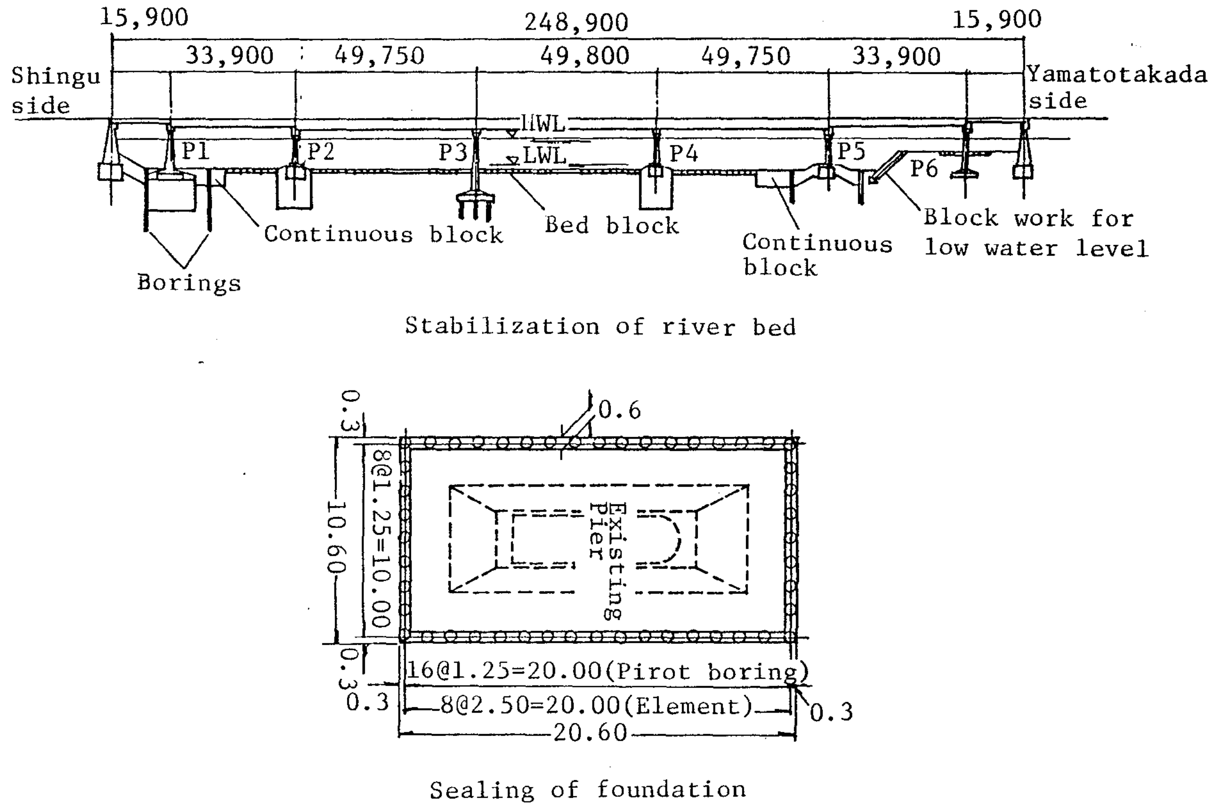
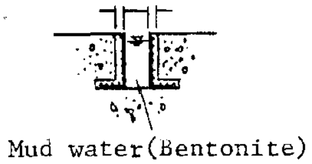
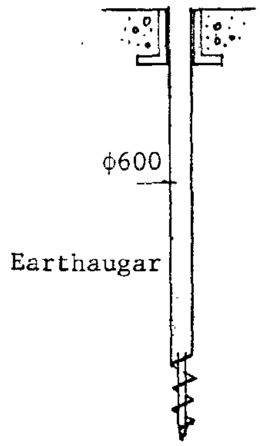


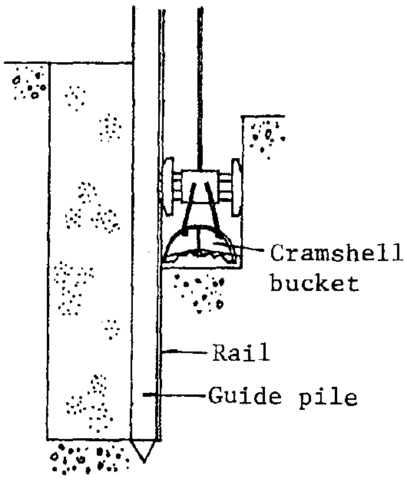
Fig. 19 Retrofitting for scouring to keep lateral resistance of foundation[10)]



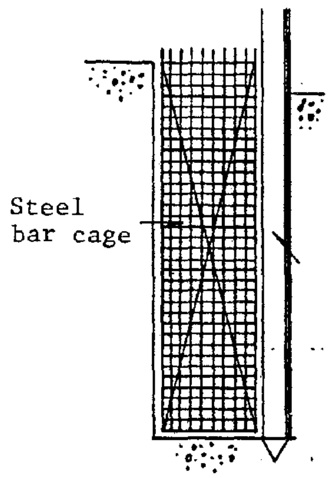
(1) Set of guidewall



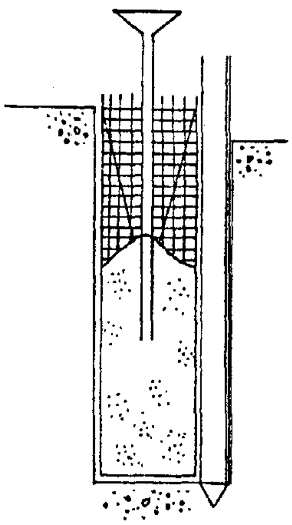
(2) Pilot boring



(3) Excavation with cramshell



(4) Hang down of steel bar



(5) Concrete works with tremie pipe

Fig. 20 Cast in site diaphragms wall[10)]

Reference

1. T. Iwasaki, Earthquake-Resistant Design of Bridges in Japan, Bulletin of Public Works Research Institute, Vol. 29, May 1973

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STRUCTURES LABORATORY AT CALIFORNIA STATE UNIVERSITY, LONG BEACH

STRUCTURES LABORATORY REPORT # SL79-11

REPORT TITLE : PRELIMINARY REPORT ON FIRE TESTING OF EPOXY REPAIRED SHEAR WALLS

AUTHORS : Joseph M. Plecnik
Associate Professor
Structures Laboratory
California State University, Long Beach
Long Beach, California 90840

Mai G. Pham
Graduate Research Engineer
Structures Laboratory
California State University, Long Beach
Long Beach, California 90840

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CHAPTER A : INTRODUCTION TO TESTING PROGRAM : PROCEDURE AND RESULTS

SEC. A.1: INTRODUCTION

This report presents the experimental test results obtained from fire tests on small-scale epoxy repaired shear wall specimens. All experiments presented herein have been conducted at the Structures Laboratory of California State University, Long Beach. Full-scale wall tests, as described by the ASTM E119 test procedure, will be performed at the University of California, Berkeley, during the summer of 1979. Although the test results presented in this report have been obtained from small-scale specimens with dimensions illustrated in Fig. A-1, the full-scale test results are not expected to vary from small-scale test results presented in subsequent chapters.

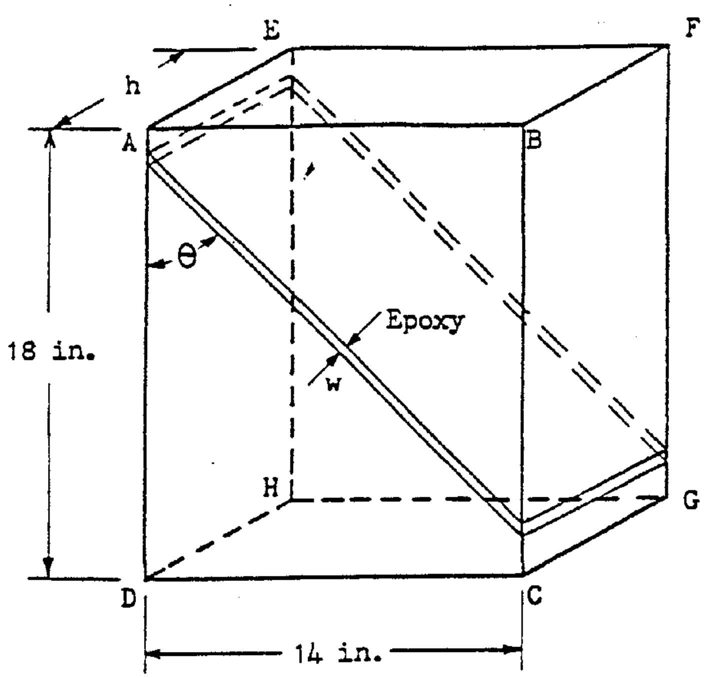

Fig. A-1 : General Specimen Configuration

SEC. A.2: CONCRETE SPECIMEN PREPARATION

Fig. A-1 provides the dimensions for all wall specimens used in this research program. The most important specimen parameters studied include wall thickness, h, and crack width, w. The specimens were constructed with wall thickness of 6 in., 8 in., and 10 in. The crack widths studied included 0.05 in., 0.10 in., and 0.25 in.

The specimens were fabricated from ready mixed concrete using a 7 bag mix. Rounded aggregate with a 3/4 in. 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 with a standard deviation of 0.36 ksi.

The shear wall specimens illustrated in Fig. A-1 were cured for approximately seven days prior to the formation of the crack. To simulate actual crack surfaces of concrete shear walls, each shear wall specimen was broken as a beam at an angle θ equal to 45°. Since compression loads were applied to the top and bottom surfaces (ABFE and CDGH in Fig. A-1), 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 and relative humidity of 50%. After the 90 day curing period, the specimens were injected with appropriate epoxy adhesives as described in Sec. A-4.

SEC. A.3: EPOXY ADHESIVES USED IN THE EXPERIMENTAL PROGRAM

Six different structural epoxy adhesives were considered in this research program. 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 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 low viscosity epoxy adhesives were obtained from four manufacturers including Delta Plastics Co., Visalia, Ca.; Hunt Process Co., Santa Fe Springs, Ca.; IPA Systems, Philadelphia, Pa.; and Adhesive Engineering, San Carlos, Ca. The range of mechanical properties for epoxy adhesives supplied by these four manufacturers are as follows:

Viscosity (cps)	300 - 800
Compressive Strength at 70°F (psi)	12,000 - 17,000
Tensile Strength at 70°F (psi)	7,000 - 12,000
Pot Life (minutes)	20 - 40
Heat Distortion Temperature (°F)	120 - 145
Strength Transition Temperature (°F)	220 - 240

Considerable variation in the strength properties of these low viscosity epoxy adhesives did not affect fire test results because the heat distortion and the strength transition temperatures were similar for all four epoxies. Hence, the test results for all four low viscosity epoxies in subsequent chapters are combined into a single group of results for low viscosity epoxy adhesives.

The high viscosity epoxy adhesives were obtained from two manufacturers; Delta Plastics Co., Visalia, Ca. and Sika Chemical Corp., Lyndhurst, New Jersey. The range of mechanical properties for two epoxy adhesives supplied by these two manufacturers are as follows:

Viscosity (cps)	12,000 - 17,000
Compressive Strength at 70°F (psi)	13,000 - 16,100
Tensile Strength at 70°F (psi)	6,500 - 7,800
Pot Life (minutes)	30 - 50
Heat Distortion Temperature (°F)	105 - 135
Strength Transition Temperature (°F)	230 - 245

Considerable variation in the strength properties of these two high viscosity epoxy adhesives did not affect fire test results because the heat distortion and the strength transition temperatures were similar for both epoxies. Hence, the test results for these two epoxy adhesives are combined in subsequent chapters into a single group of test results for high viscosity epoxy adhesives.

SEC. A.4: EPOXY INJECTION PROCEDURE AND EPOXY CURING

The epoxy resin and hardner for all six 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 ozs. with the aid of a high speed drill. The epoxy was either injected into the cracks at pressures below 100 psi 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 in Sec. A.3, 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.

SEC. A.5: DESCRIPTION OF ASTM AND SDHI FIRE EXPOSURES: HOT STRENGTH AND RESIDUAL STRENGTH

The epoxy repaired shear wall specimens described in the succeeding chapters were subjected to "pseudo-fire exposures" designed to simulate two different types of building fires. The two-hour duration ASTM E119 fire exposure 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. Both the ASTM and the SDHI time-temperature curves are provided in Fig. A-2. As indicated by the results in subsequent chapters, the ASTM

E119 type fire exposure is far more severe than the SDHI type on the fire rating of epoxy repaired structures.

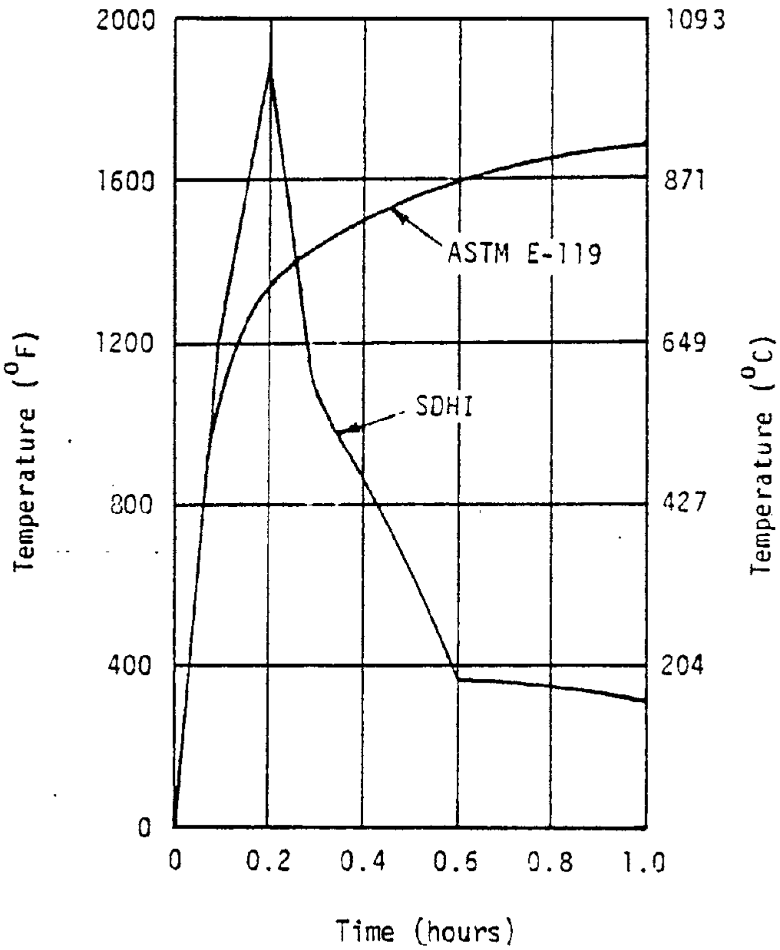

Fig. A-2: ASTM E119 and SDHI Time-Temperature Fire Curves

During fire exposure, the 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 specimens which were subjected to compression loads immediately after the fire exposure. "Residual strength" tests refer to epoxy repaired concrete specimens that were subjected to fire exposure, allowed to cool in a laboratory environment

(70°F and 50% relative humidity) for a period of seven days, and then subjected to compression loads. As indicated in later chapters "residual strengths" of epoxy repaired shear walls were significantly higher as compared to "hot strength". Sec. A.8 illustrates the behavior of pure epoxy adhesives at elevated temperatures. The strength properties of epoxy adhesives at elevated temperatures provide the explanation for the behavior of "hot strength" test results in subsequent chapters.

SEC. A.6: FIRE SURFACE COATINGS

Relatively low ultimate strength results in Chaptre B for epoxy repaired specimens subjected to ASTM E119 fire exposure prompted a search for effective fire surface coatings which would decrease the depth of epoxy burnout and increase both the hot and residual strengths. Therefore, a series of different surface coatings were applied to the fire surface for the purpose of fire protection. These surface coatings were grouped into three categories including (1) gypsum plaster, (2) thin inorganic surface coatings and (3) thin organic surface coatings.

Gypsum plaster was mixed and applied to the fire exposed surfaces according to the appropriate specifications in the 1976 UBC. Total plaster thicknesses of 1 in. (7/8 in. thick base coat with sand aggregates and 1/8 in. thick finish coat) or 3/8 in. (1/4 in. thick base coat and 1/8 in. thick finish coat) were applied to the fire exposed surfaces. The plaster was allowed to cure for at least 30 days prior to fire testing. The UBC specifies a minimum plaster thickness of 1/2 in. for fire protection. However, the 3/8 in. plaster thickness was used in this experimental program in order to determine the minimum plaster thickness which may be effective in reducing the depth of epoxy burnout.

Thin inorganic surface coatings were also applied to the fire exposed surface in thicknesses of 0.050 in. and 0.100 in. These inorganic coatings consisted of a one to one mixture on volume basis of sodium silicate and Type I Portland Cement. This inorganic coating was applied to the fire surface with a trowel and cured a minimum of seven days prior to fire exposure. The fire test results showed that this type of thin inorganic surface coatings are ineffective. Thin organic surface coatings were also applied to the fire surfaces in the form of fire resistant epoxy foams and fire retardant intumescent paints. The thickness of these coatings included 0.050 in. and 0.100 in. and were applied to the fire surface with a trowel. These inorganic surface coatings were cured for a minimum of seven days prior to fire testing. The test results for those organic surface coatings are provided in Chapter I.

SEC. A.7: DESCRIPTION OF TEST PROCEDURE

All fire tests were conducted in the forced air natural gas furnace constructed for this research program utilizing fireproof bricks. After the specimens were fully prepared, that is, the injected epoxy had been cured for a minimum of seven days and fire surface coatings applied whenever required, the specimens were placed in the furnace with only one surface (surface ABCD in Fig. A-1) exposed to the fire. During the fire exposure, loads were not applied to the specimens. Immediately after the fire exposure, the specimens were removed from the furnace and subjected to the compression load until failure in the case of "hot strength" tests. The ultimate compressive stress data provided in subsequent chapters refers to the maximum stress applied to the specimen during the complete load cycle at a loading rate similar to that specified in ASTM C39. The depth of epoxy burnout was determined for each specimen immediately after the specimen had been failed under compression loading. The "residual strength" tests were conducted according to the test procedure described in Sec. A.5. Chapter J provides the test procedure and repair procedure for specimens subjected to fire exposure, cooled at room temperature, repaired with epoxy and cement and subsequently tested in compression.

SEC. A.8: STATIC STRENGTH PROPERTIES OF LOW VISCOSITY EPOXY ADHESIVES AT ELEVATED TEMPERATURE

This section provides a brief explanation of a series of test results on low viscosity epoxy adhesives exposed to elevated temperatures and conducted at the Structures Laboratory. An electric convection oven was used for uniform temperature control and all loads were applied statically with the MTS Dynamic Testing Machine. For compressive strength tests, the test procedure including the loading rate and specimen geometry (cylinders with 1/2" diameter and 1" length) were obtained from ASTM D-695 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. A-3 illustrates the "hot test" results for static compressive strength. Beyond 400°F 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. Curve II in Fig. A-3 provides the "residual test" results for static compressive strength. For temperature exposures of up to 300°F, the "residual compressive strength" did not change appreciably. Beyond 400°F temperature exposures, the specimens usually cracked and became rubber-like resulting in low "residual" compressive strength properties. Since the compressive tests 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. A-3 especially at temperatures near and above 400°F.

Curve I in Fig. A-3 also illustrated a drastic change in the mechanical properties in the temperature range of 200°F to 250°F. Due to the sudden drop in the "hot" strength properties at a temperature of about 230°F, 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) rather than the heat distortion temperature (136°F). These results are substantiated by the thermodynamic concepts of cure or polymerization which state that the optimum post cure temperatures are near the glass transition temperature.

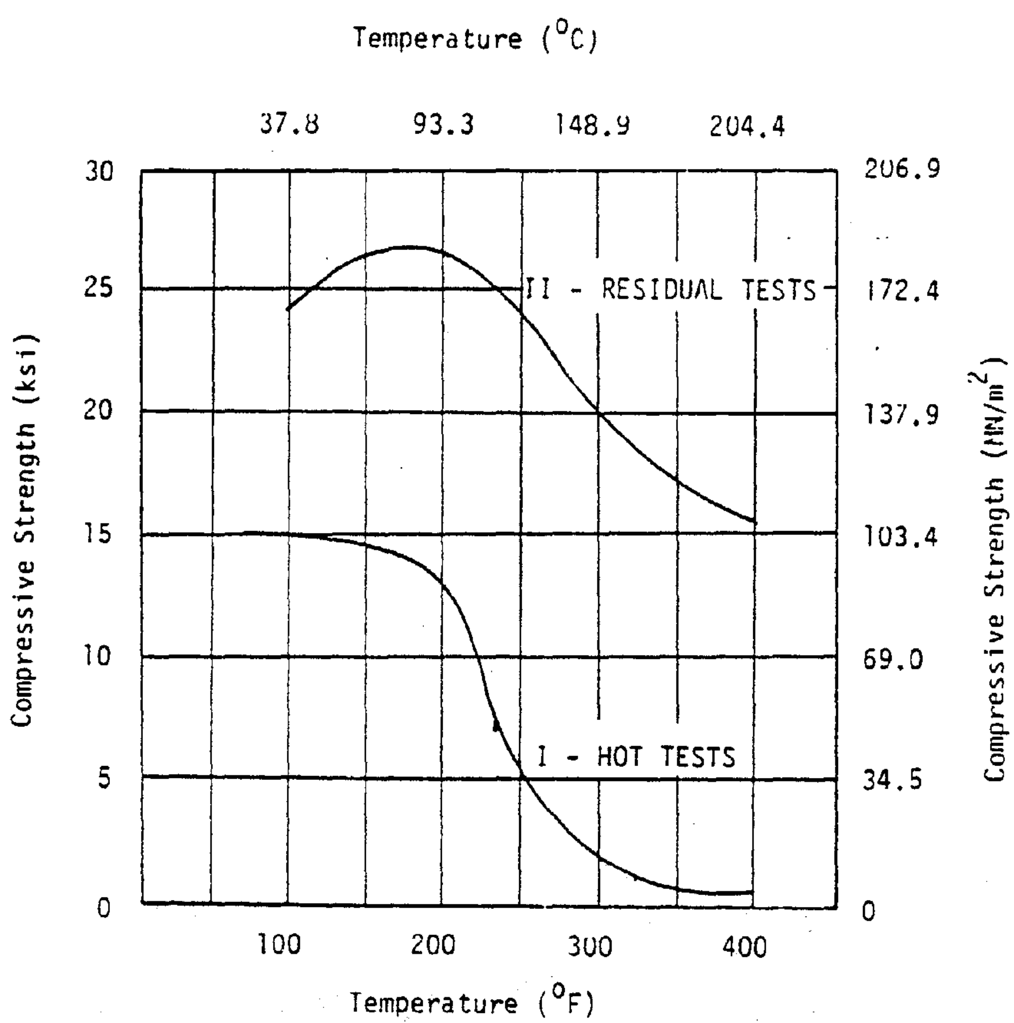

Fig. A-3: Compressive Strength of Low Viscosity Epoxy Adhesives

CHAPTER B: ASTM E119 HOT STRENGTH COMPRESSION TEST RESULTS (NO FIRE SURFACE COATING; LOW VISCOSITY EPOXY)

SEC. B.1: TEST PROCEDURE AND TEST PARAMETERS

This chapter provides a complete summary of test results for specimens whose dimensions and load application are described in Chapter A. The epoxy used to repair all cracks consisted of low viscosity type epoxies which have been described in Chapter A. All test specimens considered in this Chapter B have been exposed to the standard two-hour ASTM E119 fire exposure for walls. Primary test parameters studied in this chapter include crack widths of 0.05 in., 0.10 in. and 0.25 in. and wall thicknesses of 6 in., 8 in., and 10 in. All specimens have been subjected to ultimate compression loads immediately after the two-hour fire exposure.

SEC. B.2: SUMMARY OF TEST RESULTS

Tables B-1, B-2, and B-3 provide the test data for each specimen including the ultimate compression strength and the depth of epoxy burnout. Figs. B-1, B-2, and B-3 provide the graphical summary of average test results including average ultimate compressive stress and depth of epoxy burnout as a function of crack width. Fig. B-4 provides a complete summary of test results for average ultimate compressive stress and average depth of epoxy burnout as a function of wall thickness. Fig. B-5 provides a pictorial view of a typical failure pattern for specimens tested in this chapter. The failure pattern for all specimens, including 6 in., 8 in. and 10 in. 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 between concrete surfaces in the case of smaller crack widths. Depth of epoxy burnout is not significantly affected by crack width.

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 6 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: Low Viscosity (400 cps); Structural Grade Epoxy.

LOAD CONDITIONS: Hot strength Compression test

TIME-TEMPERATURE FIRE CURVE: ASTM E-119

TYPE OF COATING ON FIRE SURFACE: None

TABLE B-1

Specimen Number	Crack Width (inches)					
	0.05		0.10		0.25	
	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)
1	2.76	0.625	2.68	0.423	2.95	0.060
2	2.76	0.798	2.76	0.693	2.76	0.060
3	2.36	0.902	2.95	0.536	3.15	0.060
4	2.76	0.798				
Average	2.66	0.781	2.80	0.550	2.95	0.060
Standard Deviation	0.20	0.115	0.14	0.136	0.20	0.000

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The following graphs illustrate the test results provided on page B-2 in TABLE B-1 .

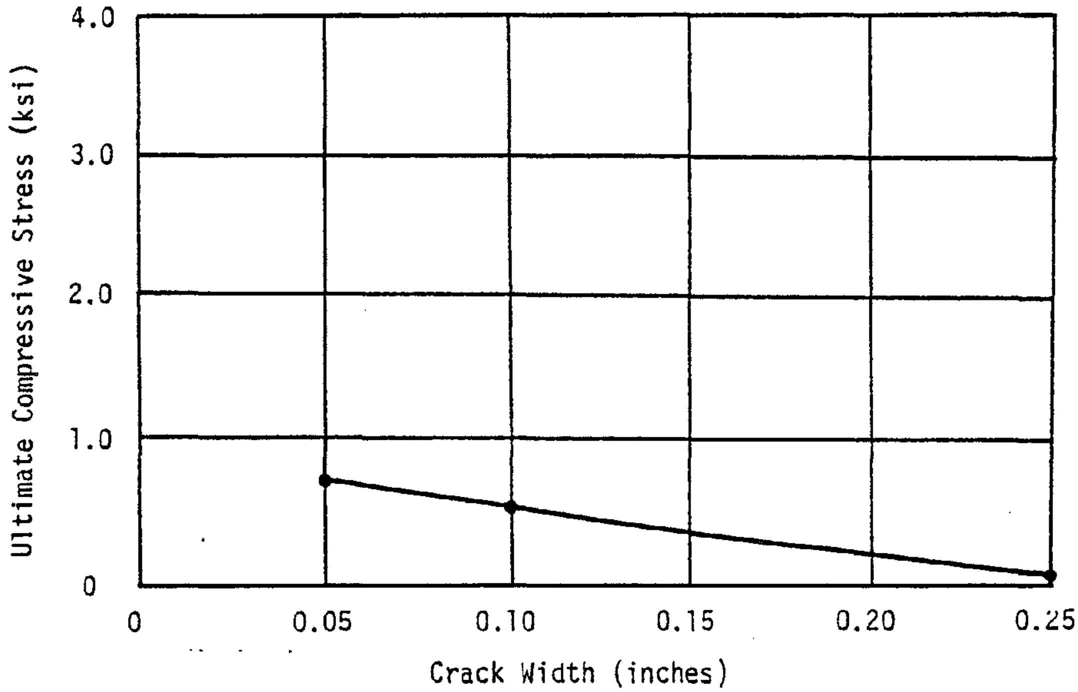

Fig. B-1a : Average Ultimate Compressive Stress as a Function of Crack Width

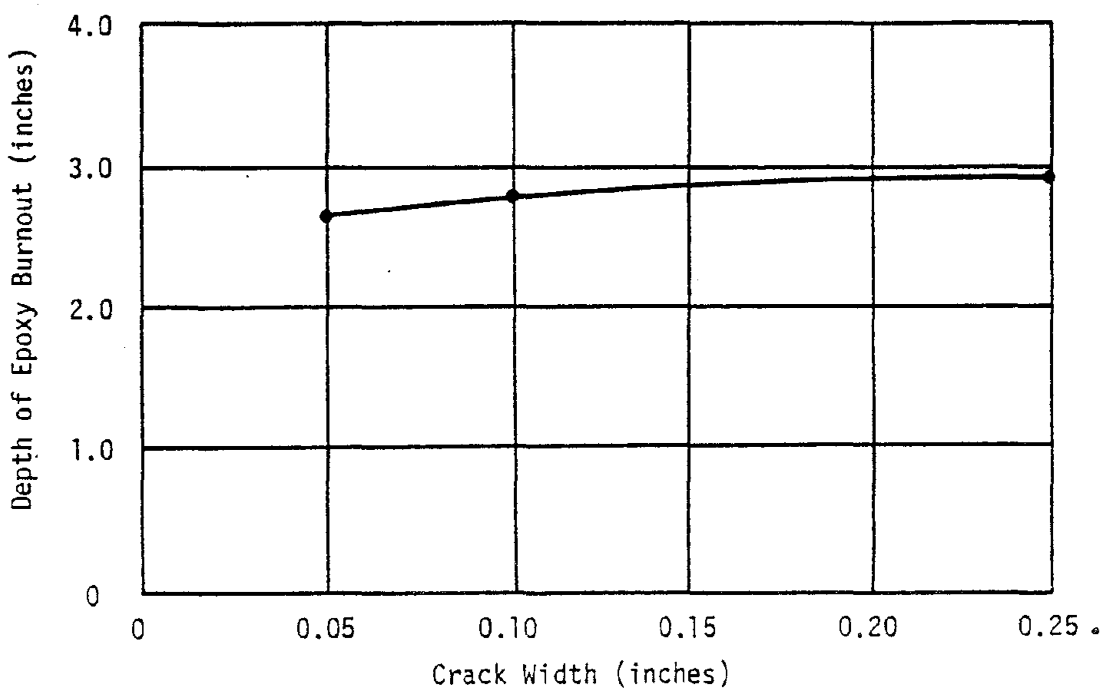

Fig. B-1b : Average Depth of Epoxy Burnout as a Function of Crack Width

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 8 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: Low Viscosity (400 cps); Structural Grade Epoxy.

LOAD CONDITIONS: Hot strength Compression test

TIME-TEMPERATURE FIRE CURVE: ASTM E-119

TYPE OF COATING ON FIRE SURFACE: NONE

TABLE B-2

Specimen Number	Crack Width (inches)					
	0.05		0.10		0.25	
	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress(ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)
1	2.56	0.830	2.76	0.481	3.35	0.146
2	2.95	0.469	2.76	0.520	3.07	0.210
3	2.36	0.729	2.76	0.491	3.15	0.182
4						
Average	2.62	0.676	2.76	0.497	3.19	0.179
Standard Deviation	0.30	0.187	0.00	0.020	0.14	0.032

The following graphs illustrate the test results provided on page B-4 in TABLE B-2.

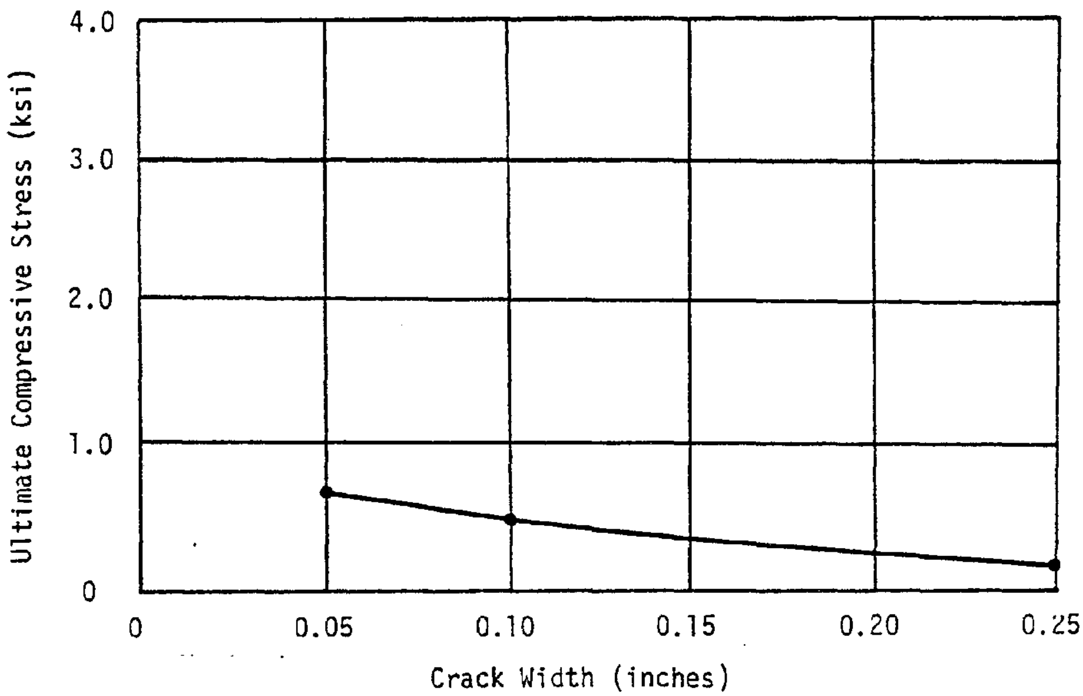

Fig. B-2a : Average Ultimate Compressive Stress as a Function of Crack Width

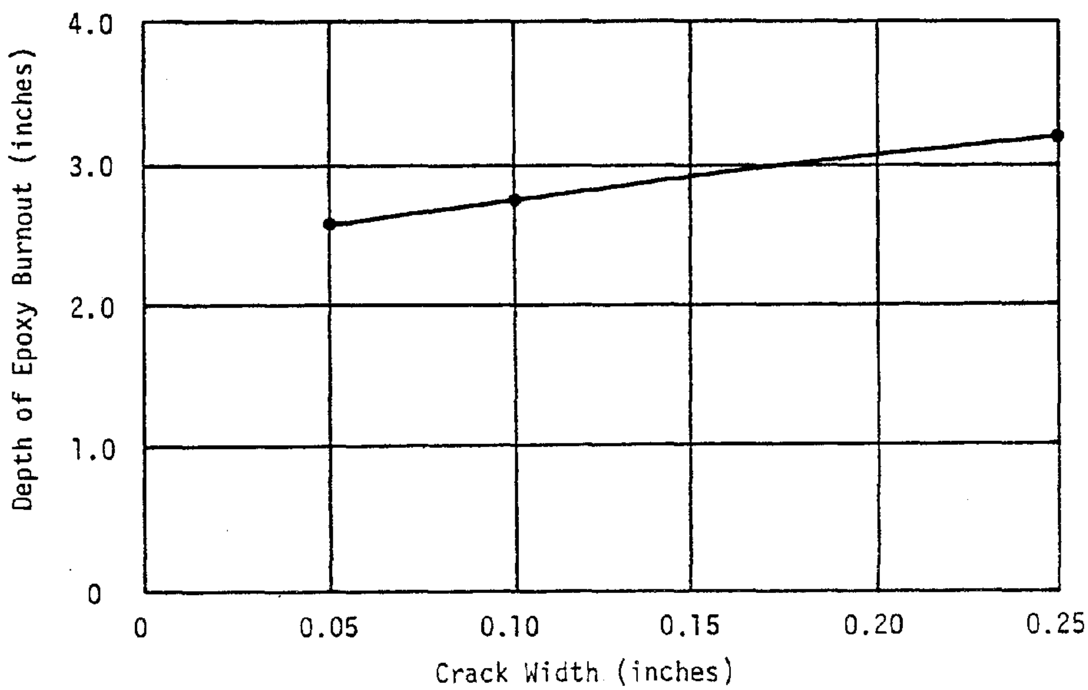

Fig. B-2b : Average Depth of Epoxy Burnout as a Function of Crack Width

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 10 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: Low Viscosity (400 cps); Structural Grade Epoxy.

LOAD CONDITIONS: Hot strength Compression test

TIME-TEMPERATURE FIRE CURVE: ASTM E-119

TYPE OF COATING ON FIRE SURFACE: None

TABLE B-3

Specimen Number	Crack Width (inches)					
	0.05		0.10		0.25	
	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress(ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)
1	1.97	1.000	2.95	0.660	2.76	0.520
2	1.77	0.877	2.91	0.510	2.95	0.430
3	1.77	0.920	2.76	0.570	2.91	0.410
4						
Average	1.84	0.936	2.87	0.582	2.87	0.455
Standard Deviation	0.11	0.062	0.10	0.078	0.10	0.057

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The following graphs illustrate the test results provided on page B-6 in TABLE B-3.

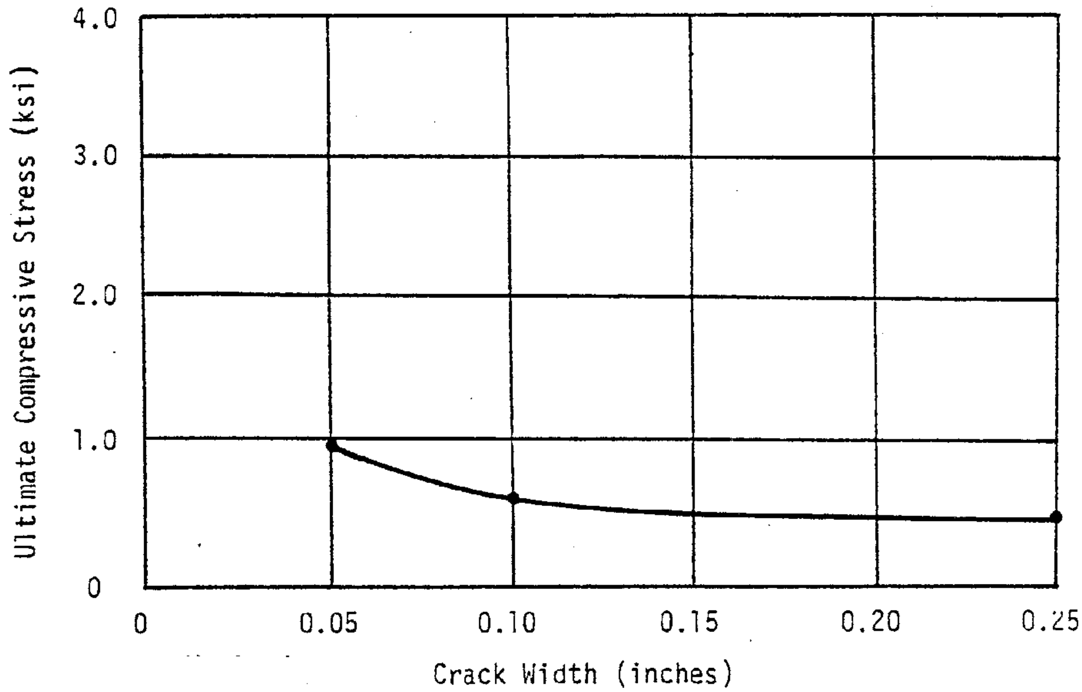

Fig. B-3a : Average Ultimate Compressive Stress as a Function of Crack Width

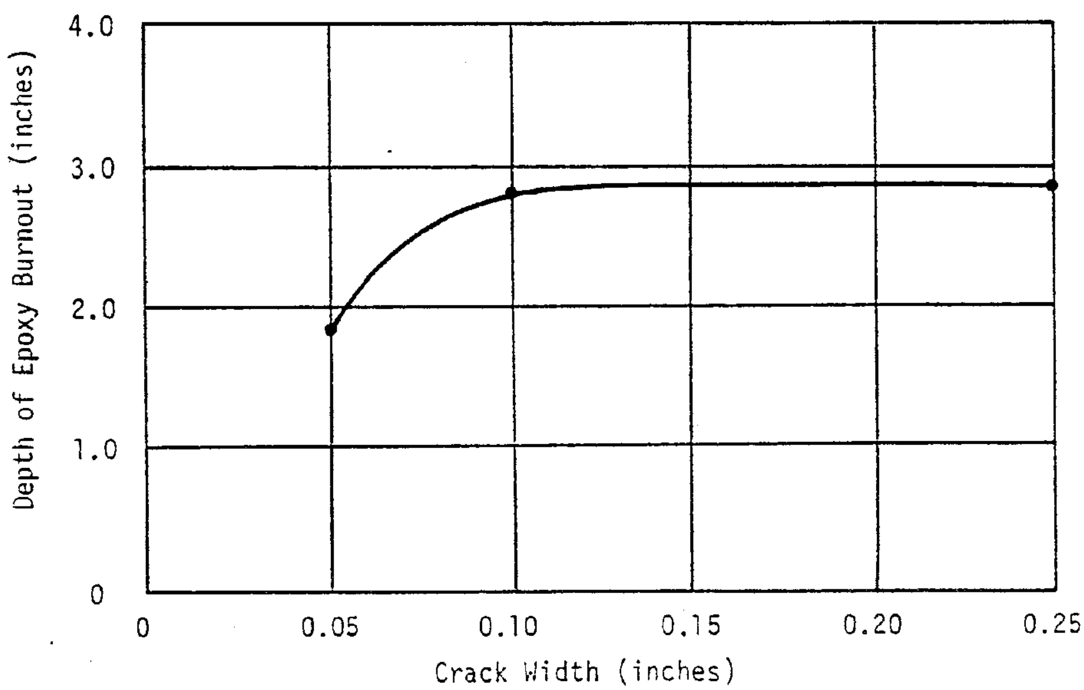

Fig. B-3b : Average Depth of Epoxy Burnout as a Function of Crack Width

These graphs illustrate the average test results provided in TABLES B-1,B-2,B-3 as a function of specimen wall thickness for various crack widths.

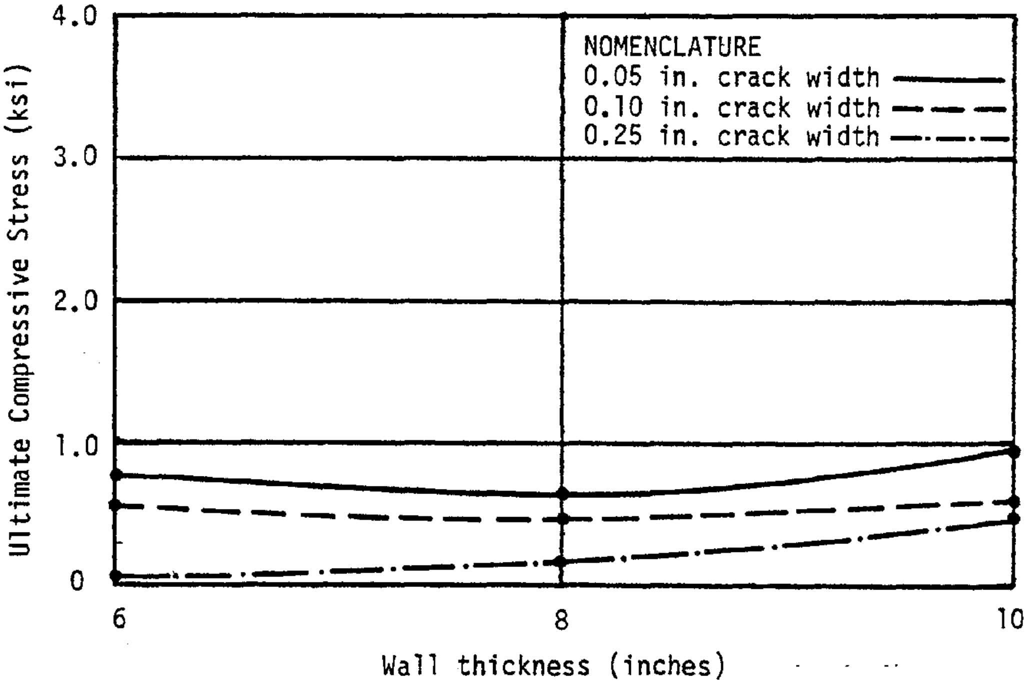

Fig. B-4a : Average Ultimate Compressive Stress as a Function of Wall Thickness

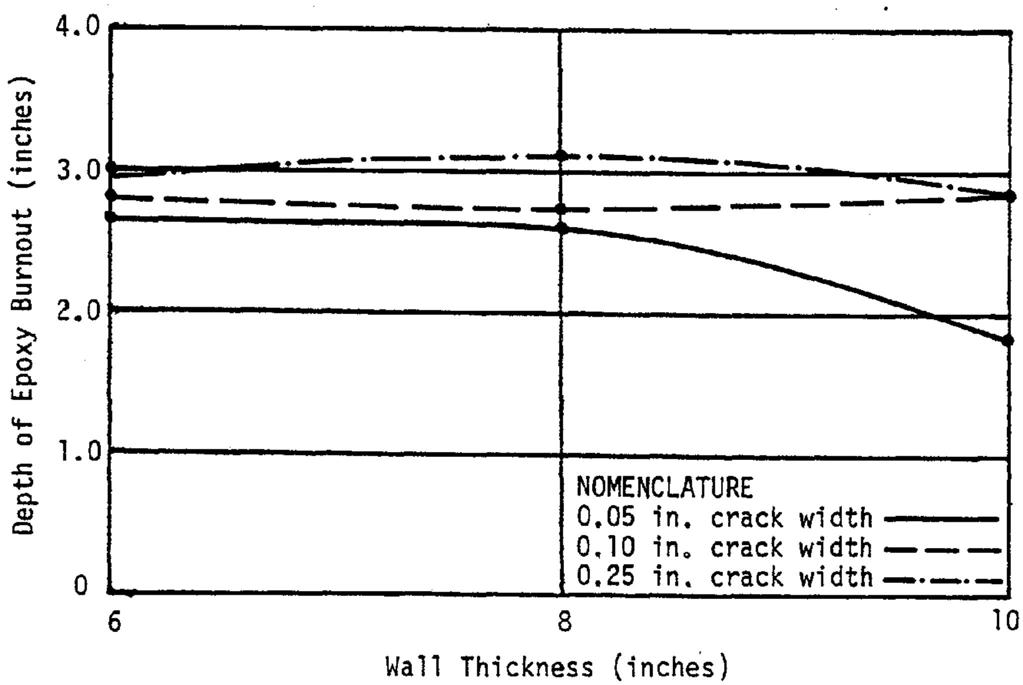

Fig. B-4b : Average Depth of Epoxy Burnout as a Function of Wall Thickness

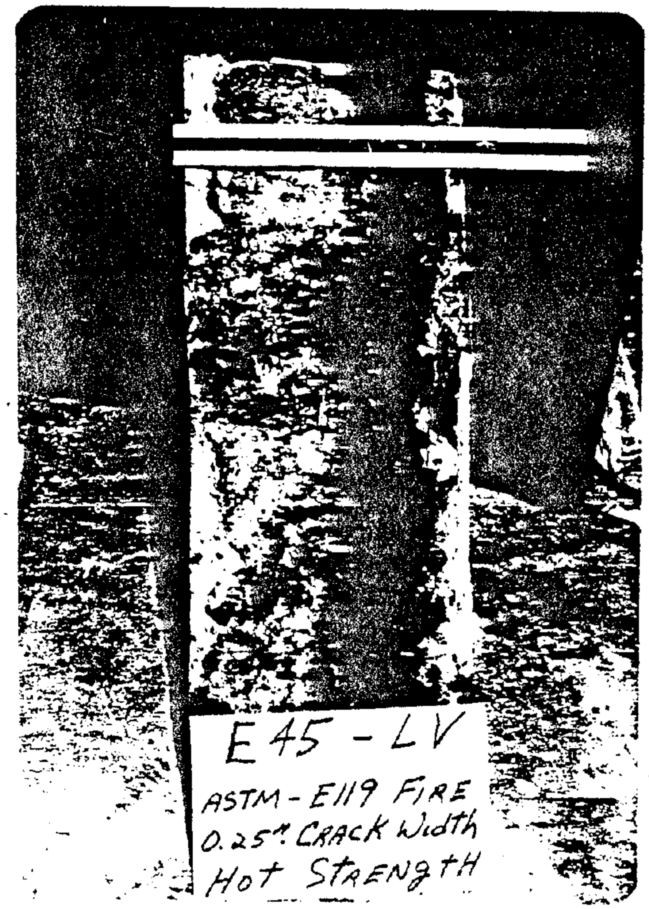

Fig. B-5 : Failure pattern for 8 in. Thick Specimen.

CHAPTER C: SDHI HOT STRENGTH COMPRESSION TEST RESULTS (NO FIRE SURFACE COATING: LOW VISCOSITY EPOXY)

SEC. C.1: TEST PROCEDURE AND TEST PARAMETERS

This chapter provides a complete summary of test results for specimens whose dimensions and load application are described in Chapter A. The epoxy used to repair all cracks consisted of low viscosity type epoxies which have been described in Chapter A. All test specimens considered in this Chapter C have been exposed to the standard SDHI fire exposure for walls. Primary test parameters studied in this chapter include crack widths of 0.05 in., 0.10 in., and 0.25 in. and wall thicknesses of 6 in., 8 in., and 10 in. All specimens have been subjected to ultimate compression loads after the fire exposure.

SEC. C.2: SUMMARY OF TEST RESULTS

Tables C-1, C-2, and C-3 provide the test data for each specimen including the ultimate compression strength and the depth of epoxy burnout. Figs. C-1, C-2, and C-3 provide the graphical summary of average test results including average ultimate compressive stress and depth of epoxy burnout as a function of crack width. Fig. C-4 provides a complete summary of test results for average ultimate compressive stress and average depth of epoxy burnout as a function of wall thickness. Fig. C-5 provides a pictorial view of a typical failure pattern for specimens tested in this chapter. Ultimate compressive stress is a function of crack width due to the development of higher frictional forces between concrete surfaces in the case of smaller crack widths. Depth of epoxy burnout is not significantly affected by crack width. The failure pattern for the 6 in. thick wall specimens consisted of shear failure in the epoxy since the temperatures within the specimens during the compression tests were above the heat distortion temperature. The failure pattern for most 8 in. and 10 in. shear wall specimens generally consisted of shear failure within concrete in regions where the epoxy was not burned out.

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 6 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: Low Viscosity (400 cps); Structural Grade Epoxy.

LOAD CONDITIONS: Hot strength Compression test

TIME-TEMPERATURE FIRE CURVE: SDHI

TYPE OF COATING ON FIRE SURFACE: None

TABLE C-1

Specimen Number	Crack Width (inches)					
	0.05		0.10		0.25	
	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress(ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)
1	0.79	0.762	1.18	0.489	1.10	0.423
2	0.69	0.920	0.98	0.850	0.98	0.658
3	0.79	0.693	0.79	0.455	0.98	0.351
4					0.87	0.524
Average	0.75	0.792	0.98	0.598	0.98	0.489
Standard Deviation	0.06	0.117	0.19	0.219	0.09	0.133

The following graphs illustrate the test results provided on page C-2 in TABLE C-1.

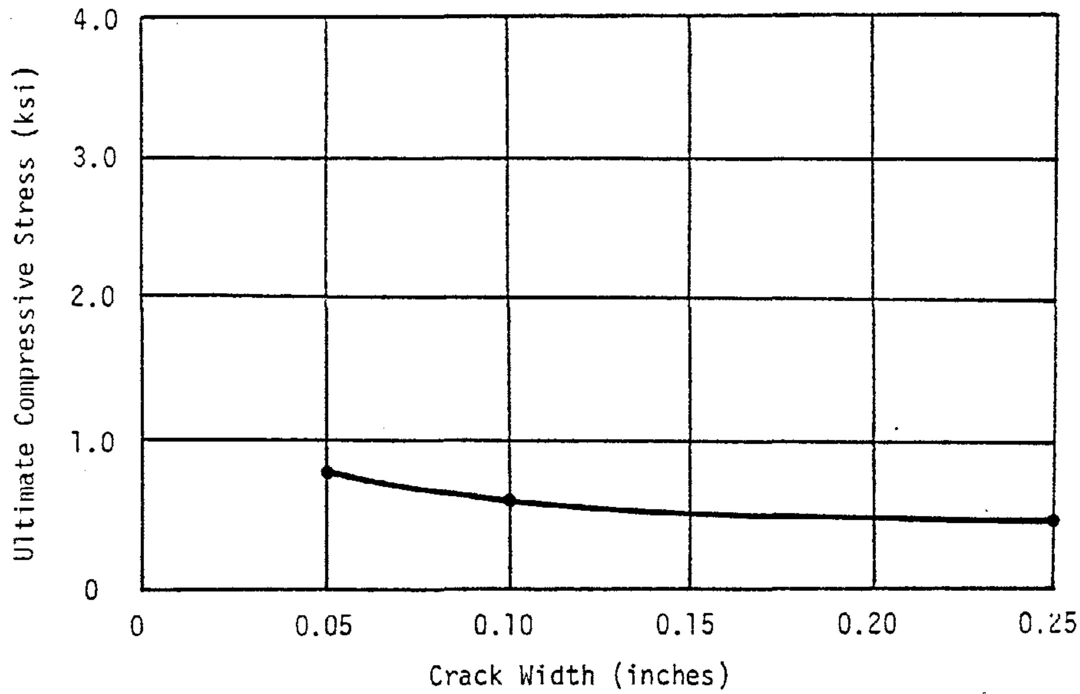

Fig. C-1a : Average Ultimate Compressive Stress as a Function of Crack Width

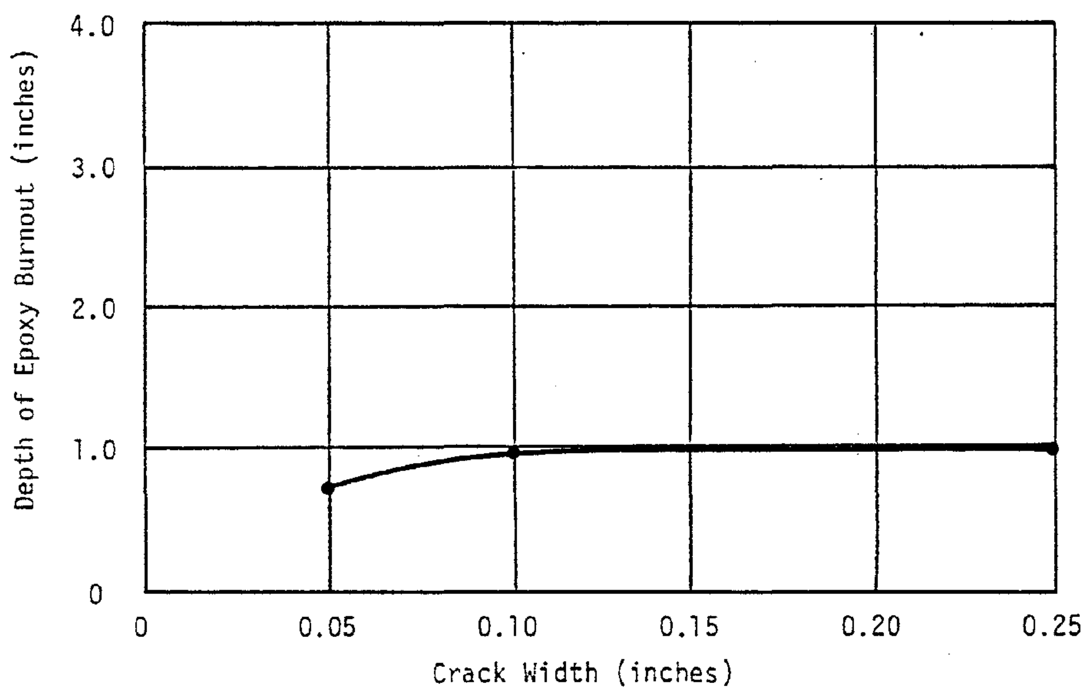

Fig. C-1b : Average Depth of Epoxy Burnout as a Function of Crack Width

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 8 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: Low Viscosity (400 cps); Structural Grade Epoxy.

LOAD CONDITIONS: Hot strength Compression test

TIME-TEMPERATURE FIRE CURVE: SDHI

TYPE OF COATING ON FIRE SURFACE: None

TABLE C-2

Specimen Number	Crack Width (inches)					
	0.05		0.10		0.25	
	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress(ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)
1	0.79	1.680	0.98	0.882	0.98	0.393
2	0.98	0.856	0.98	0.934	0.79	0.830
3	0.79	1.160	0.98	0.895	0.98	0.536
4						
Average	0.85	1.233	0.98	0.904	0.91	0.586
Standard Deviation	0.11	0.415	0.00	0.027	0.11	0.223

The following graphs illustrate the test results provided on page C-4 in TABLE C-2.

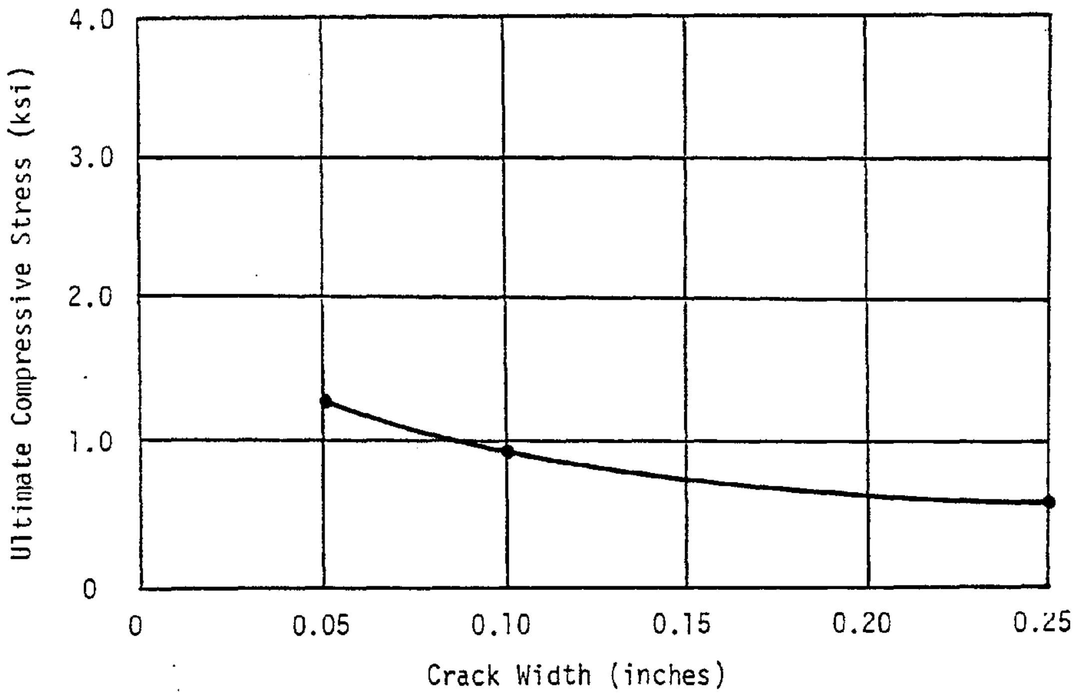

Fig.C-2a : Average Ultimate Compressive Stress as a Function of Crack Width

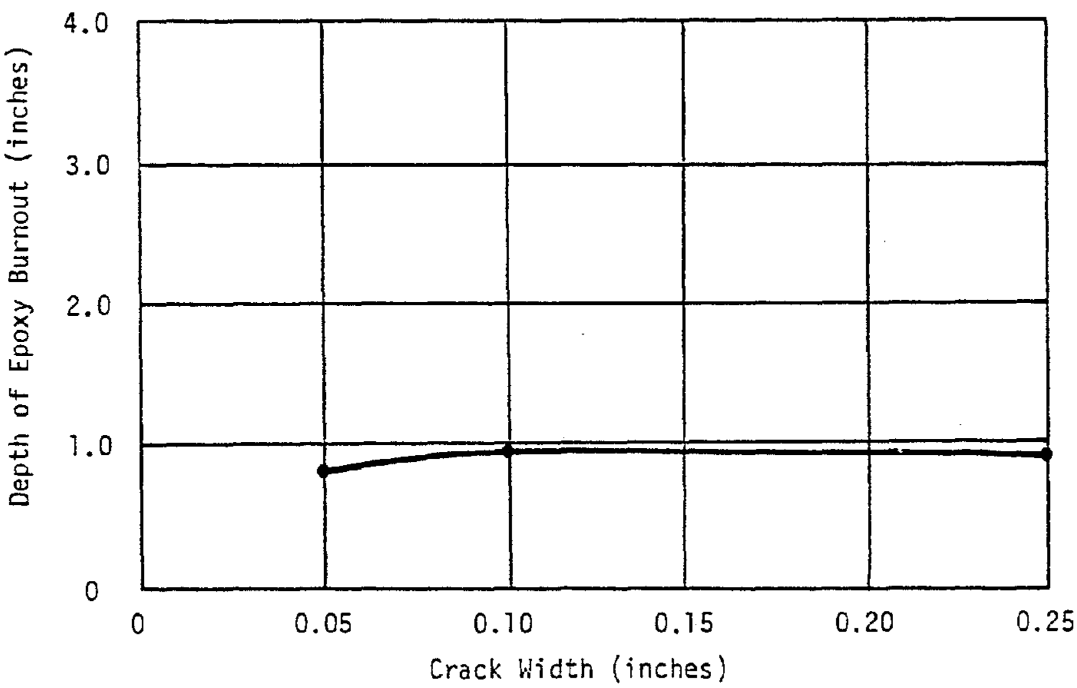

Fig.C-2b : Average Depth of Epoxy Burnout as a Function of Crack Width

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 10 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: Low Viscosity (400 cps); Structural Grade Epoxy.

LOAD CONDITIONS: Hot strength Compression test

TIME-TEMPERATURE FIRE CURVE: SDHI

TYPE OF COATING ON FIRE SURFACE: None

TABLE C-3

Specimen Number	Crack Width (inches)					
	0.05		0.10		0.25	
	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)
1	0.79	1.230	0.79	1.024	0.79	0.768
2	0.79	1.320	0.98	1.107	0.98	0.701
3	0.59	1.500	0.79	1.216	0.98	0.824
4						
Average	0.72	1.353	0.85	1.115	0.92	0.765
Standard Deviation	0.11	0.137	0.11	0.096	0.11	0.061

The following graphs illustrate the test results provided on page C-6 in TABLE C-3 .

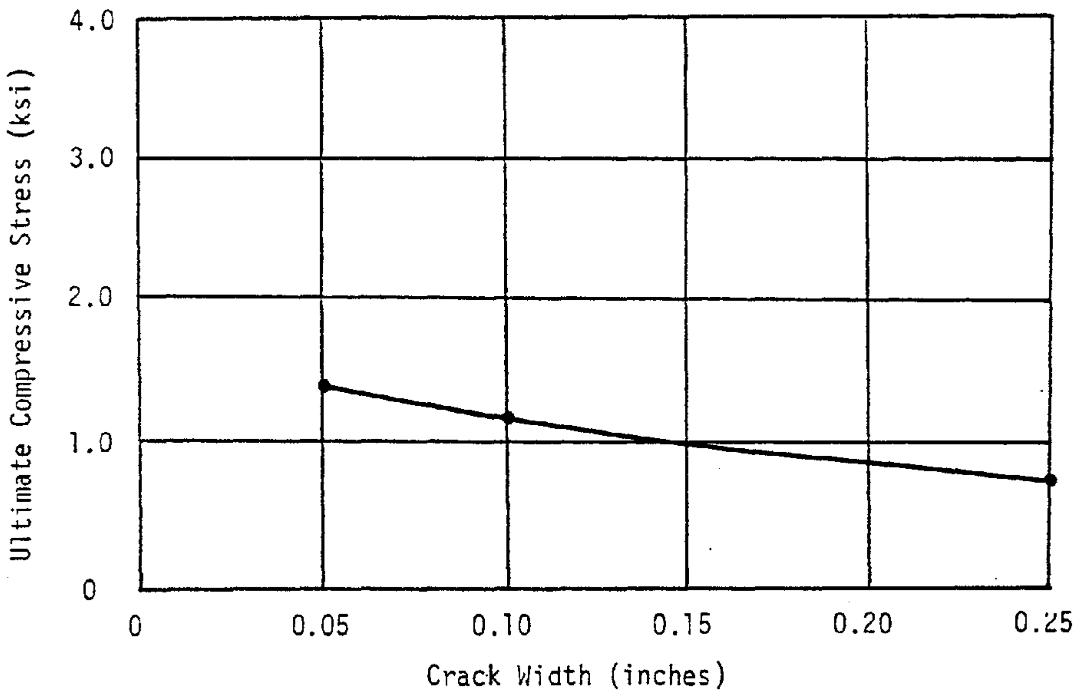

Fig. C-3a : Average Ultimate Compressive Stress as a Function of Crack Width

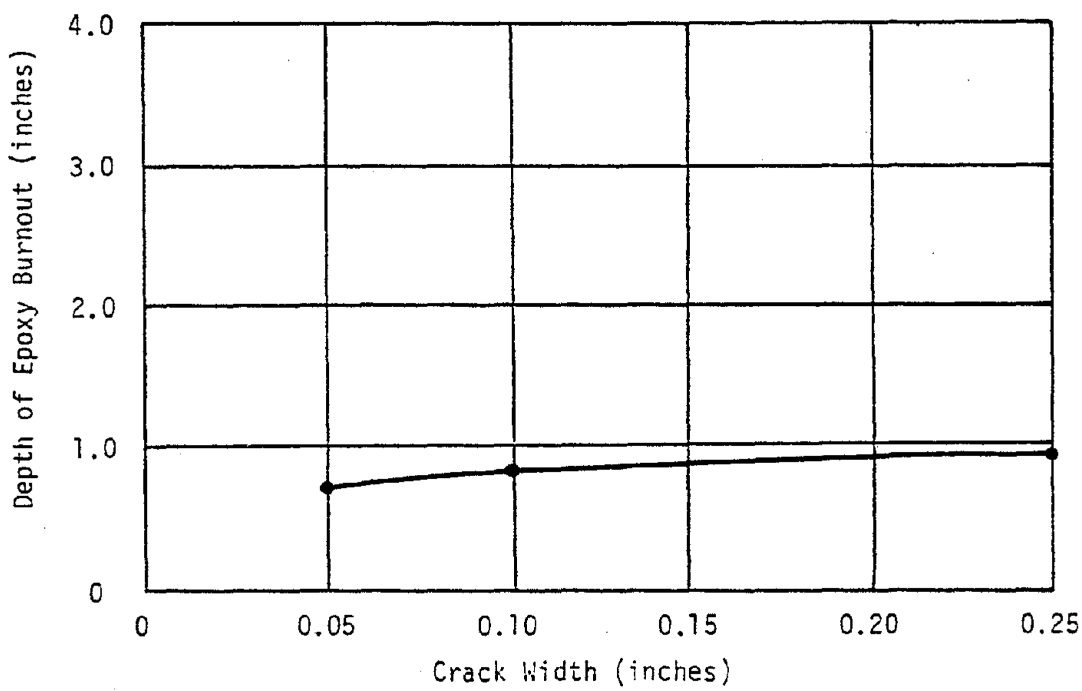

Fig.C-3b : Average Depth of Epoxy Burnout as a Function of Crack Width

STRUCTURES LABORATORY AT CALIFORNIA STATE UNIVERSITY, LONG BEACH

These graphs illustrate the average test results provided in TABLES C-1,C-2,C-3 as a function of specimen wall thickness for various crack widths.

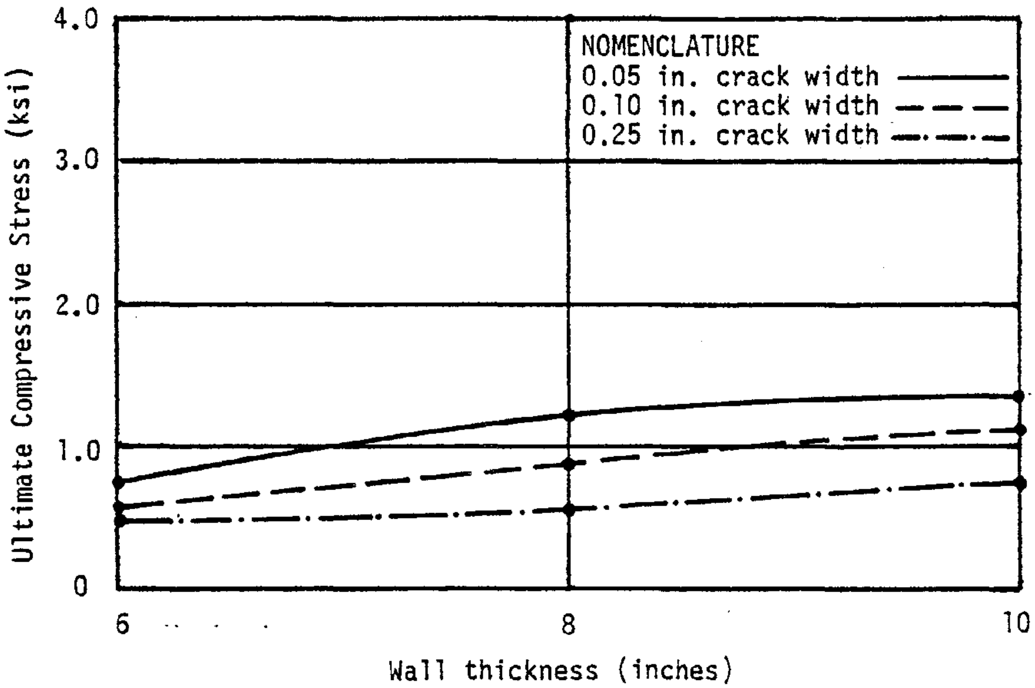

Fig. C-4a : Average Ultimate Compressive Stress as a Function of Wall Thickness

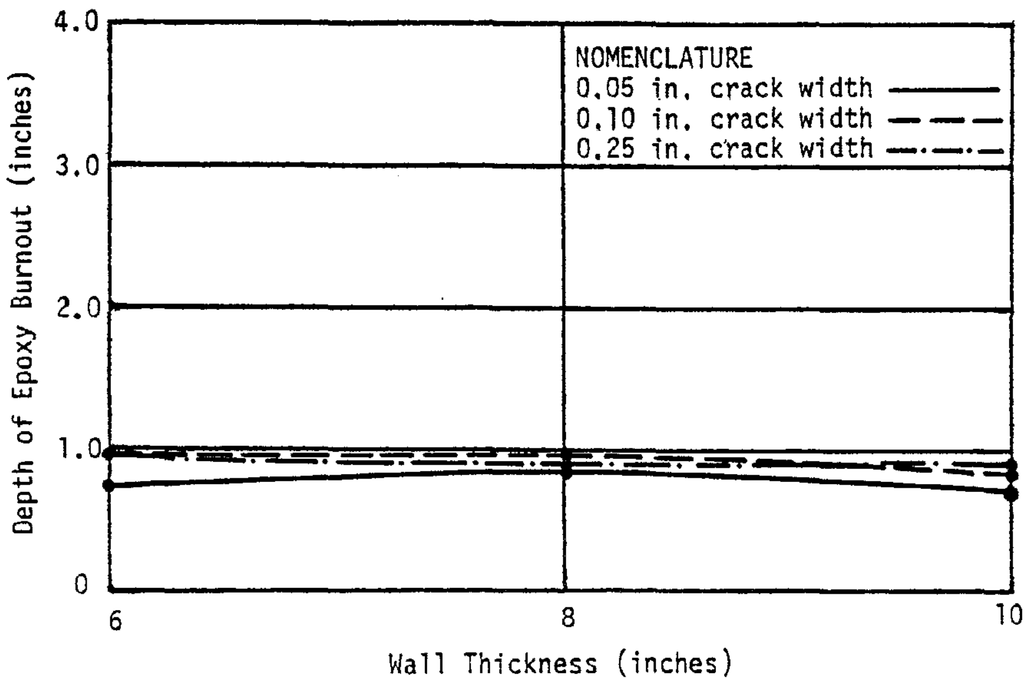

Fig. C-4b : Average Depth of Epoxy Burnout as a Function of Wall Thickness

Fig. C-5a : Failure pattern for 6 in. Thick Specimen. Note the failure through the Epoxy.

Fig. C-5b : Failure pattern for 8 in. Thick Specimen. Note the failure through the Concrete.

CHAPTER D: ASTM E119 RESIDUAL STRENGTH COMPRESSION TEST RESULTS (NO FIRE SURFACE COATING; LOW VISCOSITY EPOXY)

SEC. D.1: TEST PROCEDURE AND TEST PARAMETERS

This chapter provides a complete summary of test results for specimens whose dimensions and load application are described in Chapter A. The epoxy used to repair all cracks consisted of low viscosity type epoxies which have been described in Chapter A. All test specimens considered in the Chapter D have been exposed to the standard 2-hour ASTM E119 fire exposure for walls. Primary test parameters studied in this chapter include crack widths of 0.05 in., 0.10 in. and 0.25 in. and wall thicknesses of 6 in., 8 in., and 10 in. All specimens have been subjected to ultimate compression loads seven days after the 2-hour fire exposure.

SEC. D.2: SUMMARY OF TEST RESULTS

Tables D-1, D-2, and D-3 provide the test data for each specimen including the ultimate compression strength and the depth of epoxy burnout. Figs. D-1, D-2, and D-3 provide the graphical summary of average test results including average ultimate compressive stress and depth of epoxy burnout as a function of crack width. Fig. D-4 provides a complete summary of test results for average ultimate compressive stress and average depth of epoxy burnout as a function of wall thickness. Fig. D-5 provides a pictorial view of a typical failure pattern for specimens tested in this chapter. Ultimate compressive stress is a funciton of crack width due to the development of higher frictional forces between concrete surfaces in the case of smaller crack widths. Depth of epoxy burnout is not significantly affected by crack width. The failure pattern for the 6 in. thick wall specimens consisted of a combined shear failure in epoxy and concrete. For the 8 in. and 10 in. thick wall specimens, the failure pattern consisted usually of a shear failure in concrete.

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 6 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: Low Viscosity (400 cps); Structural Grade Epoxy.

LOAD CONDITIONS: Residual strength Compression test

TIME-TEMPERATURE FIRE CURVE: ASTM E-119

TYPE OF COATING ON FIRE SURFACE: None

TABLE D-1

Specimen Number	Crack Width (inches)					
	0.05		0.10		0.25	
	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress(ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)
1	2.75	2.533	2.95	2.060	3.15	1.726
2	2.55	2.361	2.95	2.161	2.95	1.542
3	2.75	2.304	2.55	2.137	3.54	1.313
4						
Average	2.69	2.399	2.82	2.119	3.21	1.527
Standard Deviation	0.11	0.120	0.22	0.053	0.30	0.207

STRUCTURES LABORATORY AT CALIFORNIA STATE UNIVERSITY, LONG BEACH

The following graphs illustrate the test results provided on page D-2 in TABLE D-1.

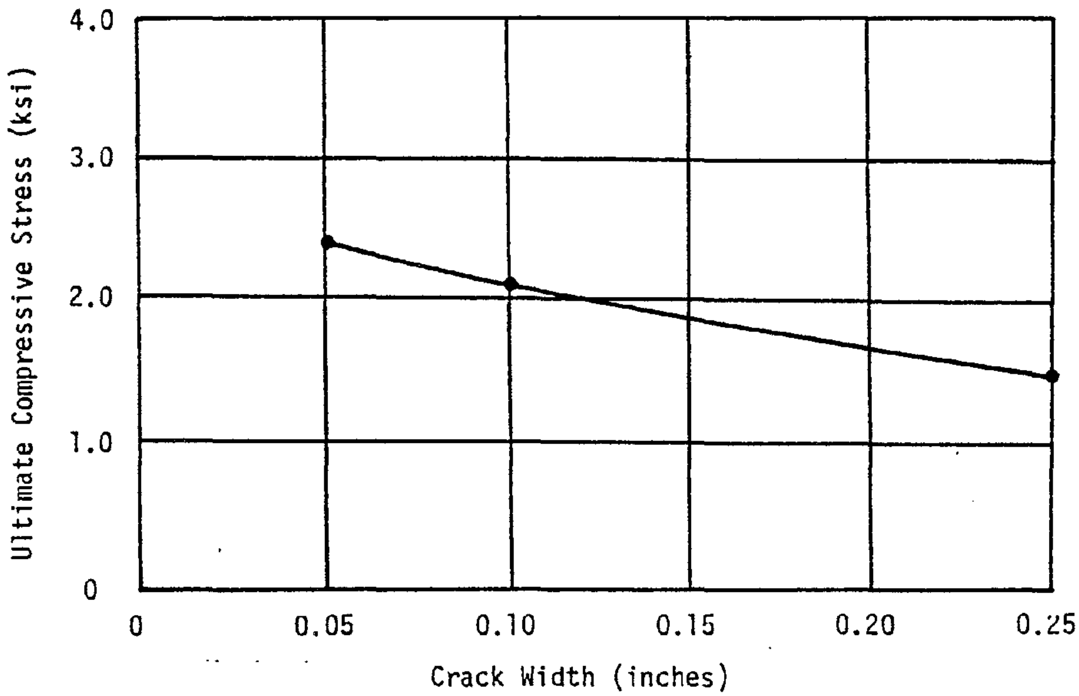

Fig. D-1a : Average Ultimate Compressive Stress as a Function of Crack Width

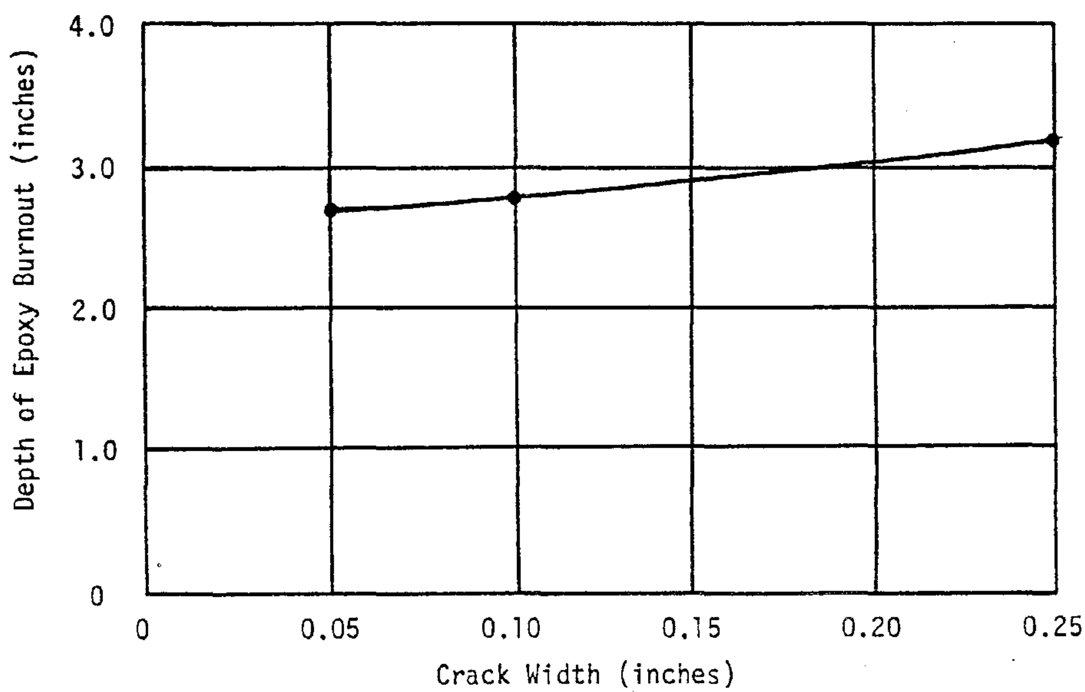

Fig. D-1b : Average Depth of Epoxy Burnout as a Function of Crack Width

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 8 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: Low Viscosity (400 cps); Structural Grade Epoxy.

LOAD CONDITIONS: **Residual strength Compression test**

TIME-TEMPERATURE FIRE CURVE: **ASTM E-119**

TYPE OF COATING ON FIRE SURFACE: **None**

TABLE D-2

Specimen Number	Crack Width (inches)					
	0.05		0.10		0.25	
	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)
1	2.48	2.445	2.75	2.496	2.75	2.290
2	2.75	2.679	3.15	2.375	3.34	2.366
3	2.75	2.634	2.75	2.592	3.26	2.084
4						
Average	2.66	2.586	2.88	2.488	3.12	2.247
Standard Deviation	0.15	0.124	0.22	0.100	0.32	0.146

STRUCTURES LABORATORY AT CALIFORNIA STATE UNIVERSITY, LONG BEACH

The following graphs illustrate the test results provided on page D-4 in TABLE D-2 .

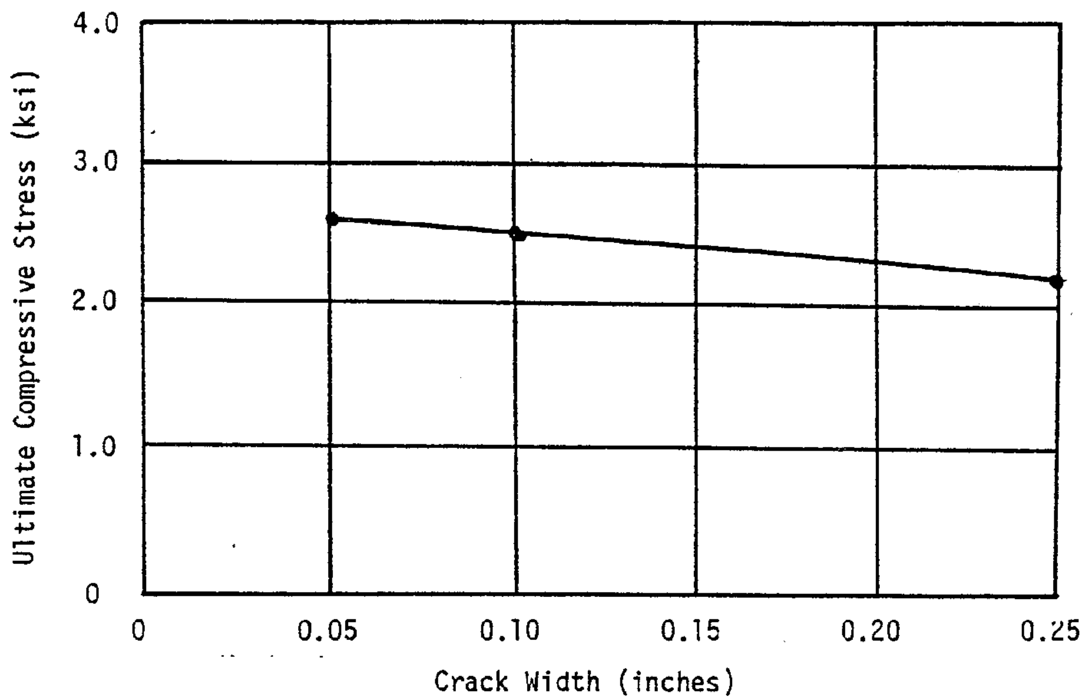

Fig. D-2a : Average Ultimate Compressive Stress as a Function of Crack Width

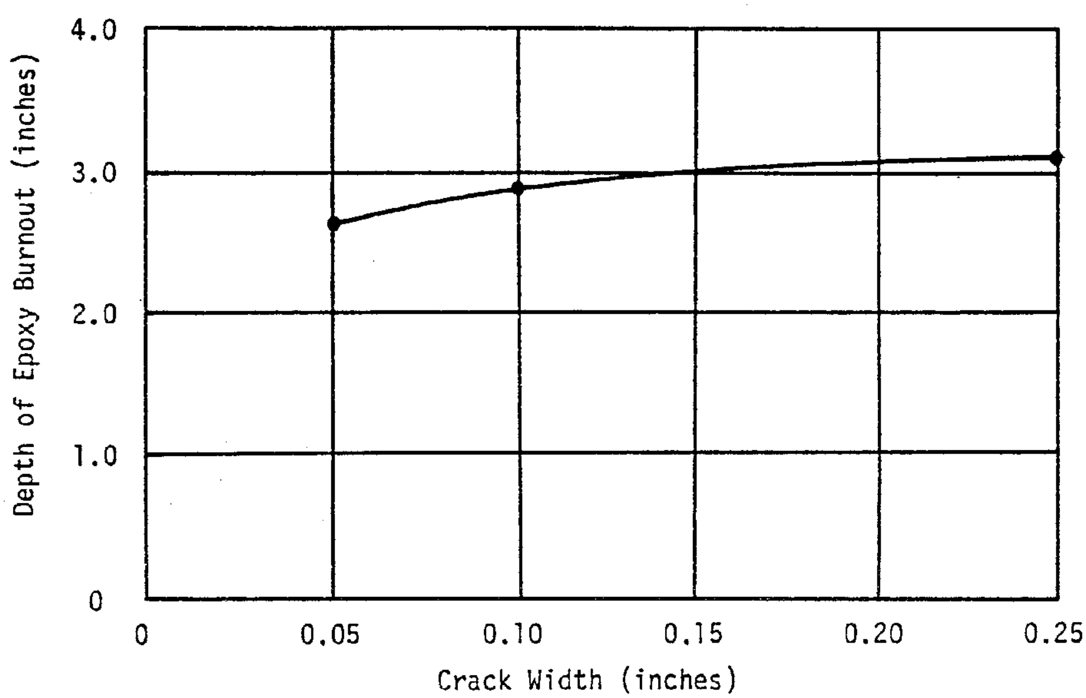

Fig. D-2b : Average Depth of Epoxy Burnout as a Function of Crack Width

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 10 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: Low Viscosity (400 cps); Structural Grade Epoxy.

LOAD CONDITIONS: Residual strength Compression test

TIME-TEMPERATURE FIRE CURVE: ASTM E-119

TYPE OF COATING ON FIRE SURFACE: None

TABLE D-3

Specimen Number	Crack Width (inches)					
	0.05		0.10		0.25	
	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)
1	2.30	*	3.00	*	3.00	*
2	1.95	*	2.50	*	3.00	*
3	2.00	*	2.80	*	2.90	*
4						
Average	2.08		2.77		2.97	
Standard Deviation	0.19		0.25		0.06	

* The strength of this specimen was above the 300.00 Kips (2.14 Ksi) capacity of the experimental testing equipment.

These graphs illustrate the average test results provided in TABLES D-1,D-2,D-3 as a function of specimen wall thickness for various crack widths.

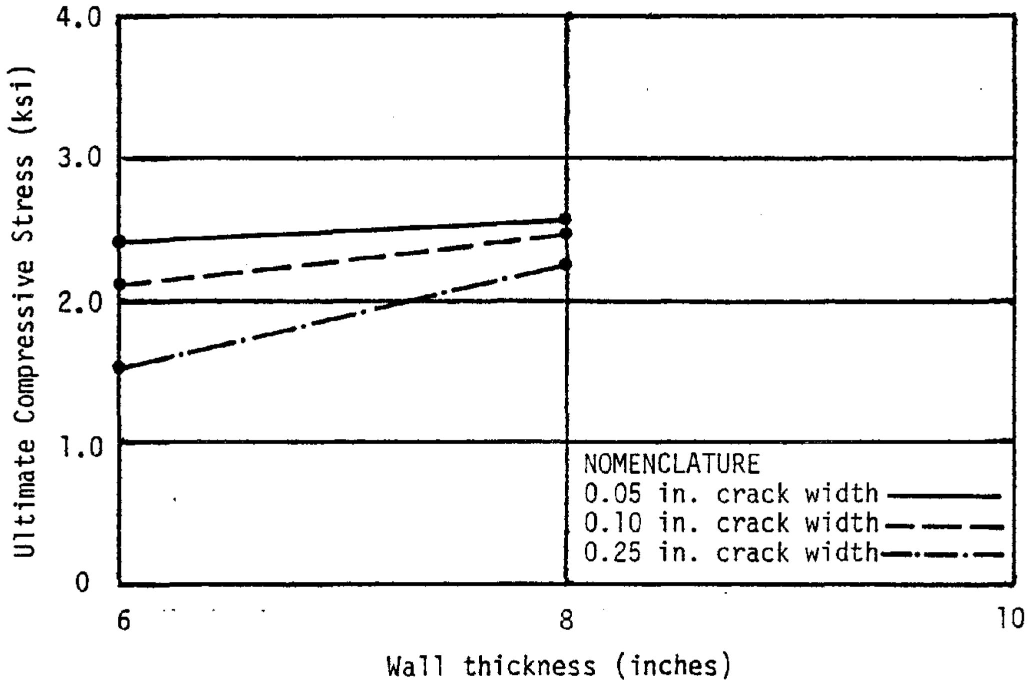

Fig. D-4a : Average Ultimate Compressive Stress as a Function of Wall Thickness

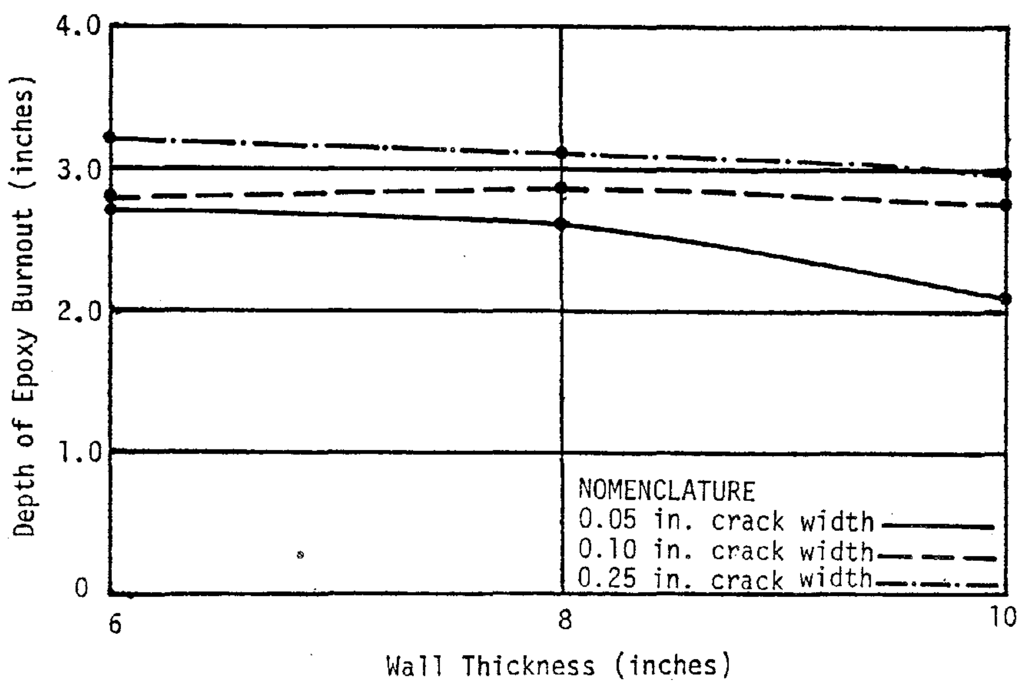

Fig. D-4b : Average Depth of Epoxy Burnout as a Function of Wall Thickness

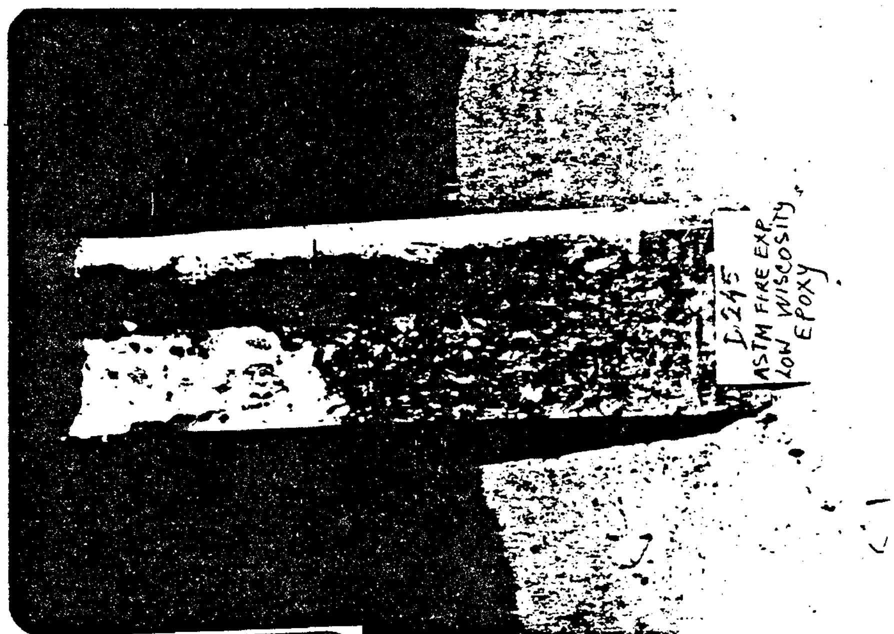

Fig. D-5a : Failure pattern for 6 in. Thick Specimen. Note the Combined Shear failure through Epoxy and Concrete

Fig. D-5b : Failure pattern for 8 in Thick Specimen. Note the Shear failure through Concrete not through Epoxy.

CHAPTER E: SDHI RESIDUAL STRENGTH COMPRESSION TEST RESULTS (NO FIRE SURFACE COATING; LOW VISCOSITY EPOXY)

SEC. E.1: TEST PROCEDURE AND TEST PARAMETERS

This chapter provides a complete summary of test results for specimens whose dimensions and load application are described in Chapter A. The epoxy used to repair all cracks consisted of low viscosity type epoxies which have been described in Chapter A. All test specimens considered in this Chapter E have been exposed to the standard SDHI fire exposure for walls. Primary test parameters studied in this chapter include crack widths of 0.05 in., 0.10 in. and 0.25 in. and wall thicknesses of 6 in., 8 in., and 10 in. All specimens have been subjected to ultimate compression loads seven days after the fire exposure.

SEC. E.2: SUMMARY OF TEST RESULTS

Tables E-1, E-2, and E-3 provide the test data for each specimen including the ultimate compression strength and the depth of epoxy burnout. Figs. E-1, E-2, and E-3 provide the graphical summary of average test results including average ultimate compressive stress and depth of epoxy burnout as a function of crack width. Fig. E-4 provides a complete summary of test results for average ultimate compressive stress and average depth of epoxy burnout as a function of wall thickness. Fig. E-5 provides a pictorial view of a typical failure pattern for specimens tested in this chapter. The failure pattern for all specimens, including 6 in., 8 in. and 10 in. shear wall specimens, consisted of shear failure in the concrete. Depth of epoxy burnout is not significantly affected by crack width.

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 6 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: Low Viscosity (400 cps); Structural Grade Epoxy.

LOAD CONDITIONS: Residual strength Compression test

TIME-TEMPERATURE FIRE CURVE: SDHI

TYPE OF COATING ON FIRE SURFACE: None

TABLE E-1

Specimen Number	Crack Width (inches) 0.05 Burnout (inches)	0.05 Ultimate Compressive Stress (ksi)	0.10 Burnout (inches)	0.10 Ultimate Compressive Stress(ksi)	0.25 Burnout (inches)	0.25 Ultimate Compressive Stress (ksi)
1	0.78	3.452	0.98	3.256	0.98	3.343
2	0.78	*	0.78	3.571	0.98	2.452
3	0.70	*	0.78	2.690	0.98	2.780
4	0.78	3.548				
Average	0.76	3.536	0.85	3.173	0.98	2.858
Standard Deviation	0.03	0.057	0.11	0.446	0.00	0.450

* The strength of this specimen was above the 300.0 Kips (3.57 Ksi) capacity of the experimental testing equipment.

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The following graphs illustrate the test results provided on page E-2 in TABLE E-1 .

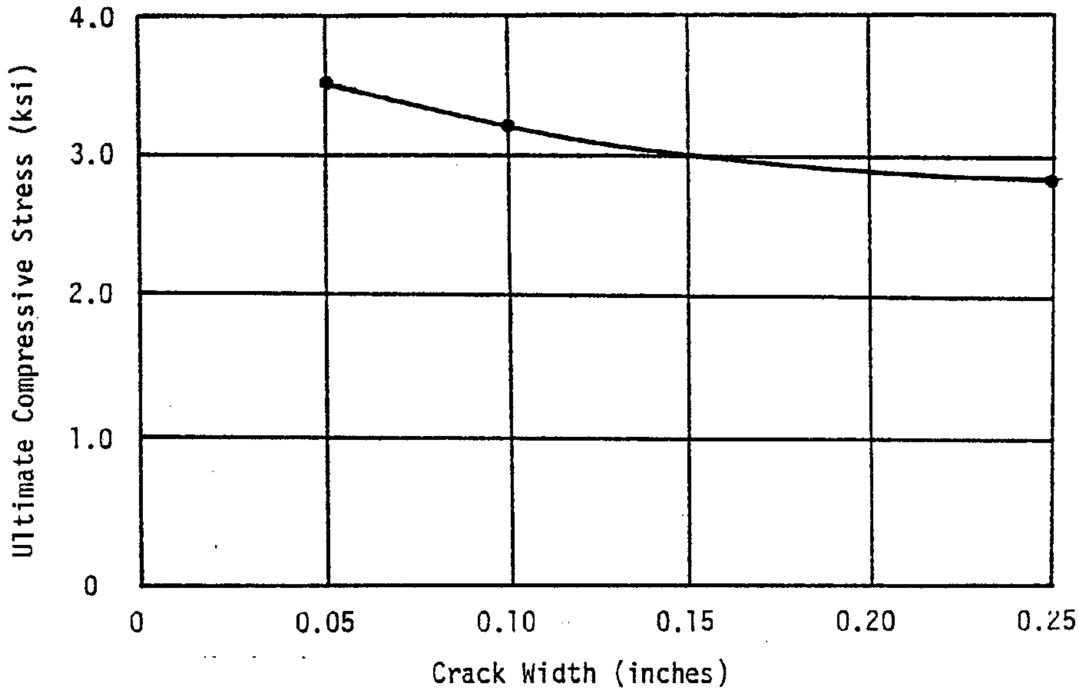

Fig. E-1a : Average Ultimate Compressive Stress as a Function of Crack Width

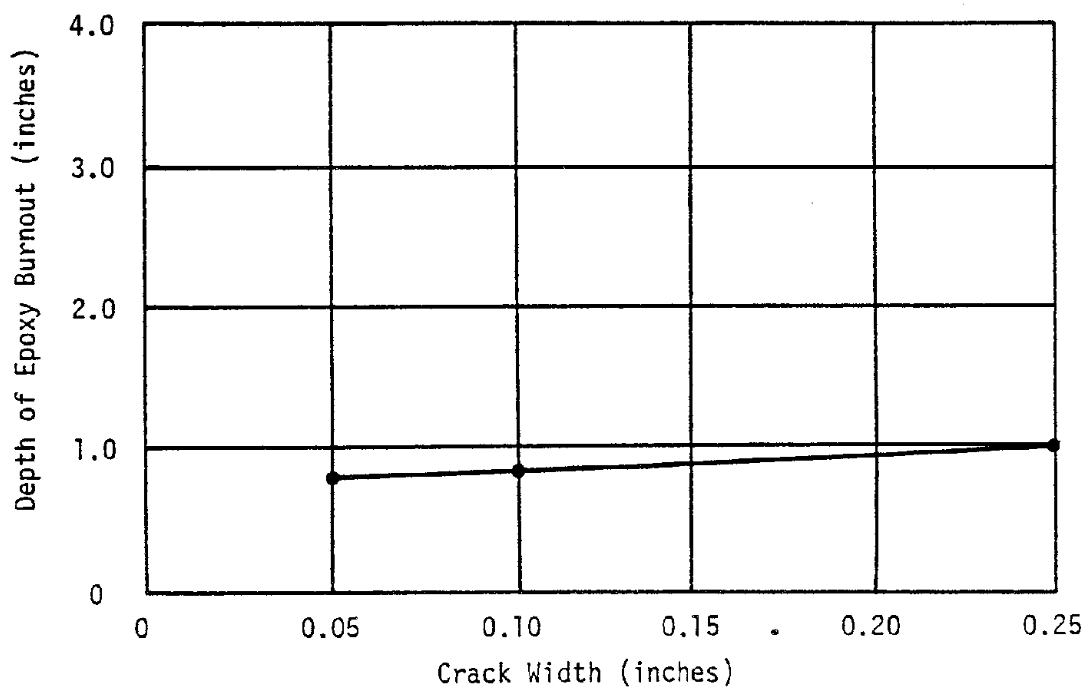

Fig. E-1b : Average Depth of Epoxy Burnout as a Function of Crack Width

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 8 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: Low Viscosity (400 cps); Structural Grade Epoxy.

LOAD CONDITIONS: Residual strength Compression test

TIME-TEMPERATURE FIRE CURVE: SDHI

TYPE OF COATING ON FIRE SURFACE: None

TABLE E-2

Specimen Number	Crack Width (inches)					
	0.05		0.10		0.25	
	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)
1	0.78	*	1.18	*	0.98	*
2	0.78	*	0.98	*	1.10	2.634
3	1.06	*	0.78	*	0.78	*
4						
Average	0.87	*	0.98	*	0.95	*
Standard Deviation	0.15		0.19		0.15	

* The strength of this specimen was above the 300.00 Kips (2.68 Ksi) capacity of the experimental testing equipment.

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The following graphs illustrate the test results provided on page E-4 in TABLE E-2 .

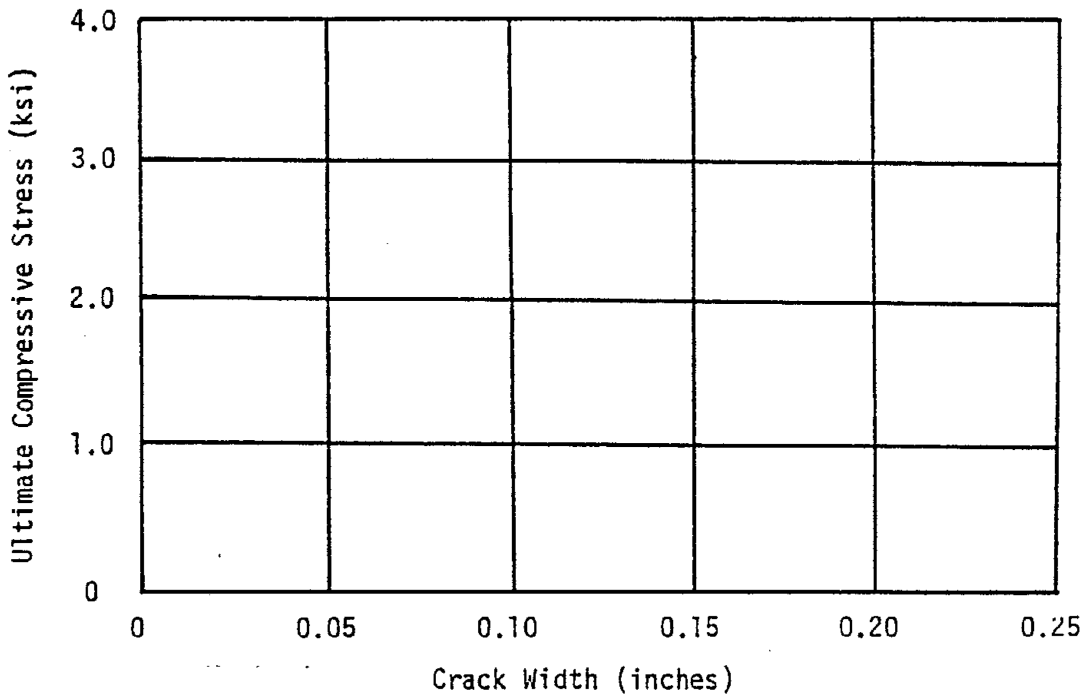

Fig.E-2a : Average Ultimate Compressive Stress as a Function of Crack Width

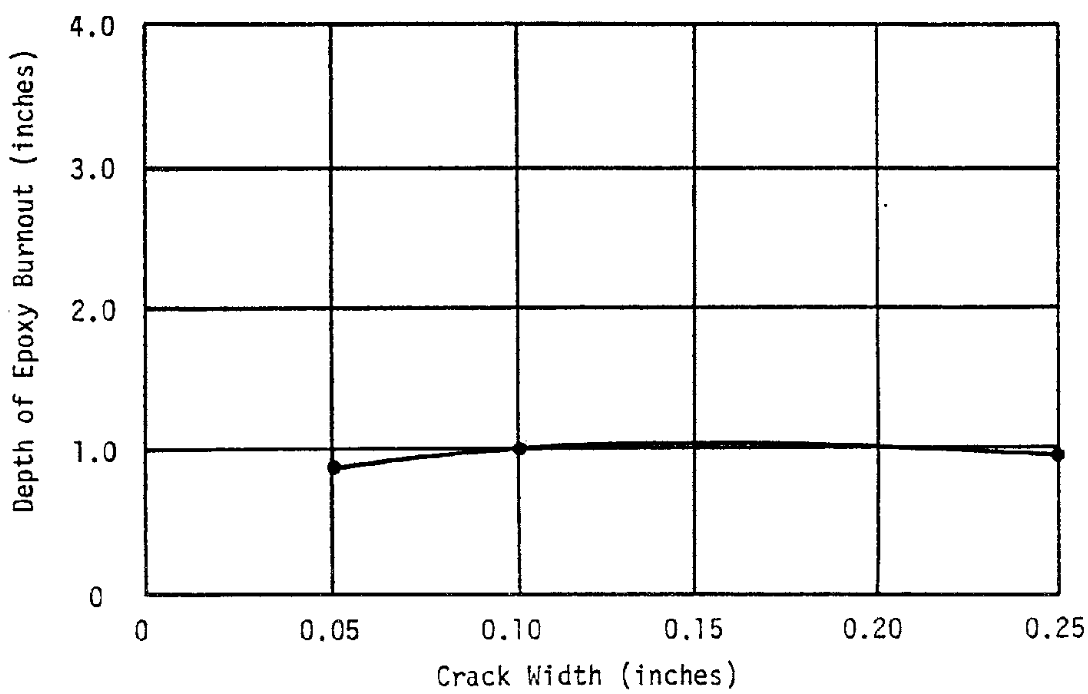

Fig.E-2b : Average Depth of Epoxy Burnout as a Function of Crack Width

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 10 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: Low Viscosity (400 cps); Structural Grade Epoxy.

LOAD CONDITIONS: Residual strength Compression test

TIME-TEMPERATURE FIRE CURVE: SDHI

TYPE OF COATING ON FIRE SURFACE: None

TABLE E-3

Specimen Number	Crack Width (inches)					
	0.05		0.10		0.25	
	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)
1	0.78	*	0.78	*	0.98	*
2	0.78	*	0.98	*	0.98	*
3	0.78	*	0.98	*	0.98	*
4						
Average	0.78	*	0.91	*	0.98	*
Standard Deviation	0.00		0.11		0.00	

* The strength of this specimen was above the 300.00 Kips (2.14 Ksi) capacity of the experimental testing equipment.

STRUCTURES LABORATORY AT CALIFORNIA STATE UNIVERSITY, LONG BEACH

The following graphs illustrate the test results provided on page E-6 in TABLE E-3 .

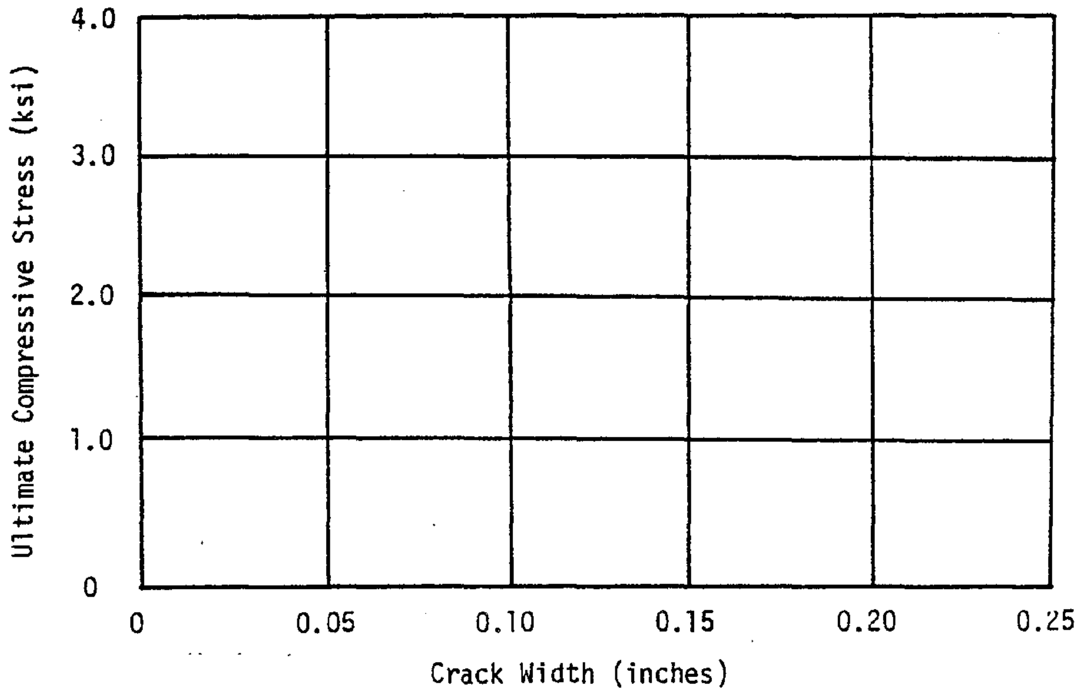

Fig.E-3a : Average Ultimate Compressive Stress as a Function of Crack Width

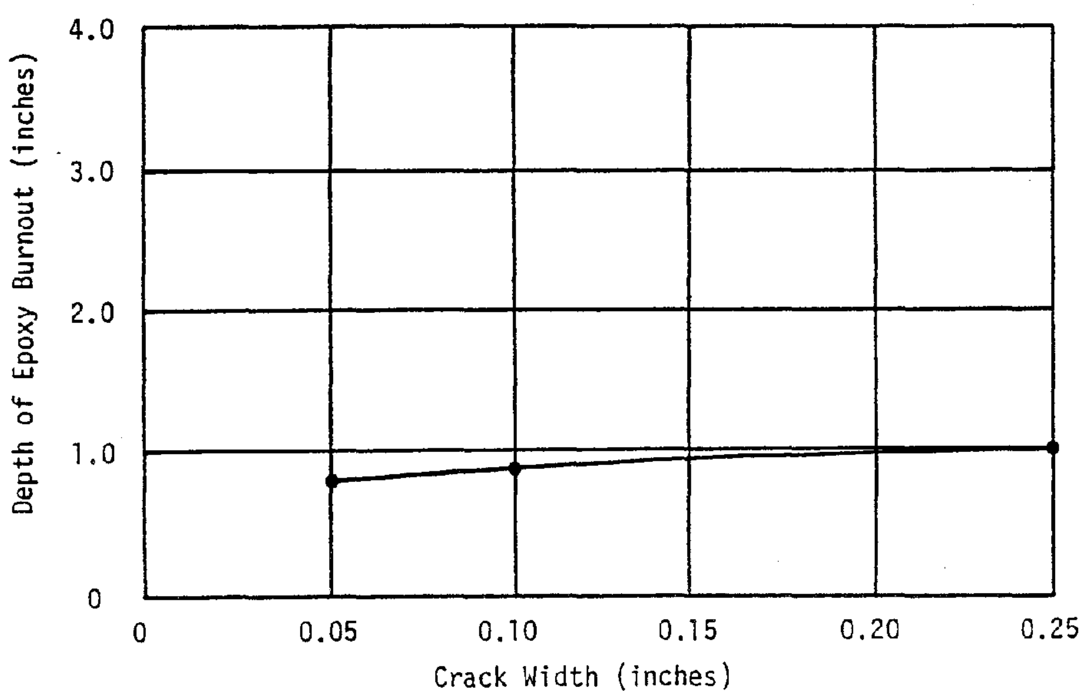

Fig.E-3b : Average Depth of Epoxy Burnout as a Function of Crack Width

These graphs illustrate the average test results provided in TABLES E-1,E-2,E-3 as a function of specimen wall thickness for various crack widths.

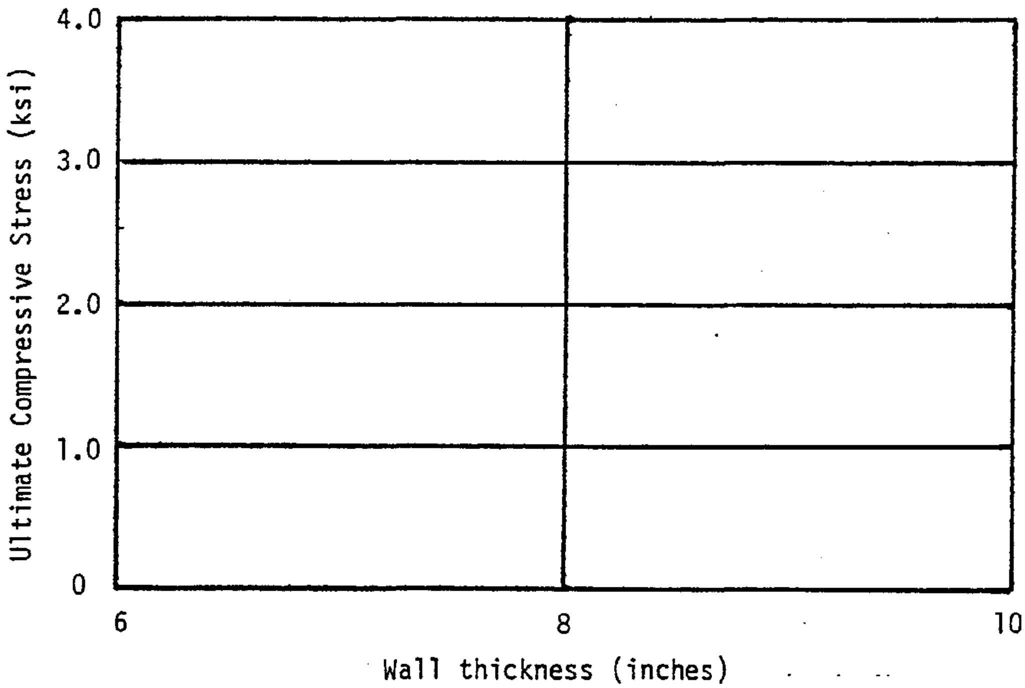

Fig. E-4a : Average Ultimate Compressive Stress as a Function of Wall Thickness

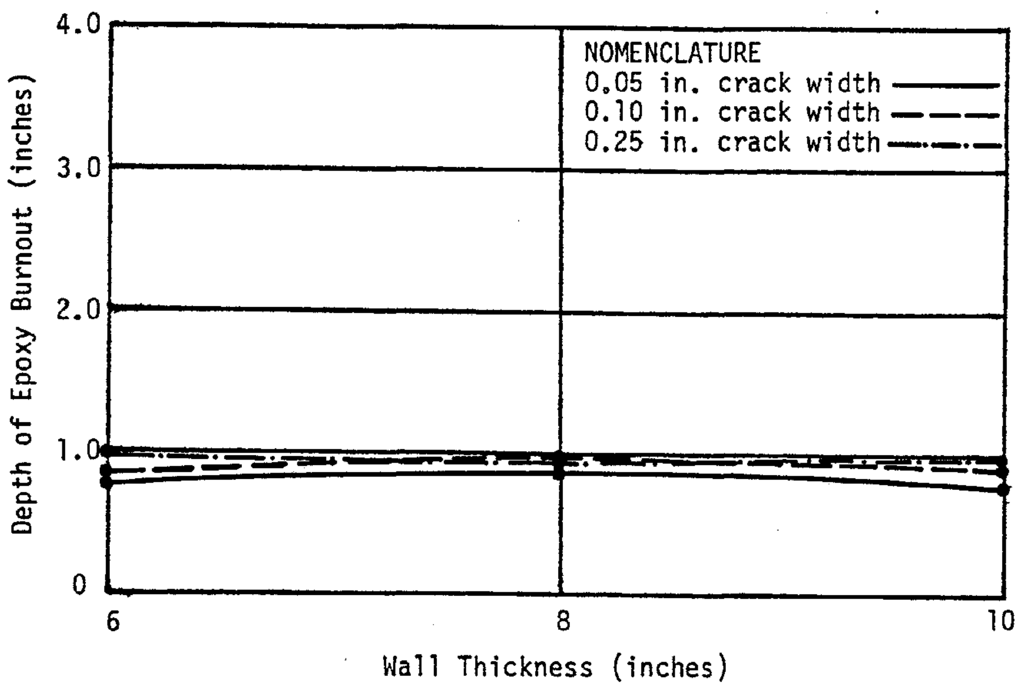

Fig. E-4b : Average Depth of Epoxy Burnout as a Function of Wall Thickness

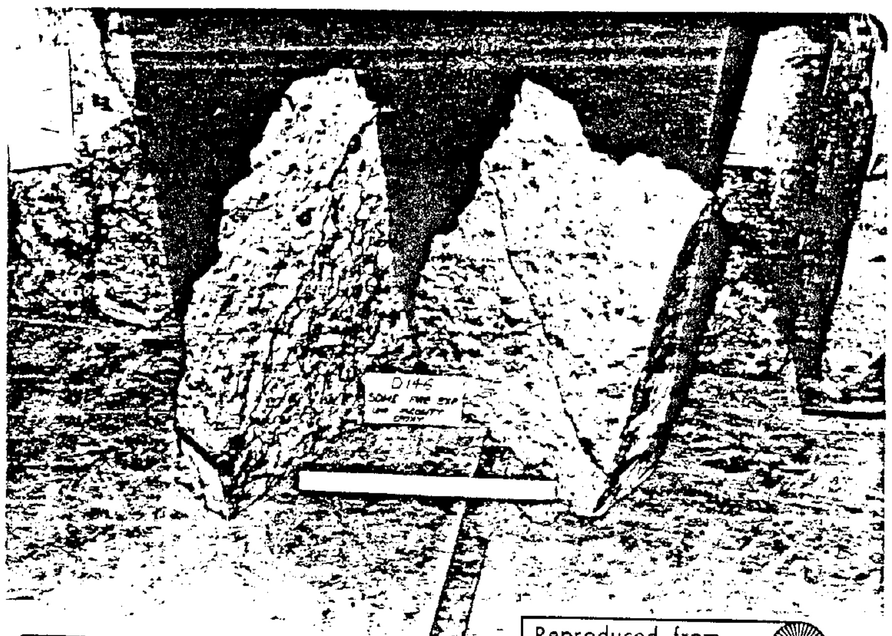

Fig. E-5 : Failure pattern for 6 in. Thick Specimen. Note the Shear failure through the Concrete.

CHAPTER F: ASTM E119 AND SDHI HOT STRENGTH COMPRESSION TEST RESULTS (NO FIRE SURFACE COATING; HIGH VISCOSITY EPOXY)

SEC. F.1: TEST PROCEDURE AND TEST PARAMETERS

This chapter provides a complete summary of test results for specimens whose dimensions and load application are described in Chapter A. The epoxy used to repair all cracks consisted of high viscosity type epoxies which have been described in Chapter A. Specimens considered in the Chapter F have been exposed to the standard 2-hour ASTM E119 or SDHI fire exposure for walls. Primary test parameters studied in this chapter include crack widths of 0.05 in., 0.10 in. and 0.25 in. and wall thickness of 6 in. All specimens have been subjected to ultimate compression loads immediately after the fire exposure.

SEC. F.2: SUMMARY OF TEST RESULTS

Tables F-1 and F-2 provide the test data for each specimen including the ultimate compression strength and the depth of epoxy burnout. Figs. F-1 and F-2 provide the graphical summary of average test results including average ultimate compressive stress and depth of epoxy burnout as a function of crack width. Fig. F-3 provides a pictorial view of a typical failure pattern for specimens tested in this chapter. The failure pattern for all specimens, including both ASTM E119 and SDHI fire exposure 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 between concrete surfaces in the case of small crack widths. Depth of epoxy burnout is not significantly affected by crack width. Comparison with results in Chapters B and C indicates that the low and high viscosity epoxies considered in this research program provide very similar results.

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 6 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: High Viscosity (15,000 cps); Structural Grade Epoxy

LOAD CONDITIONS: Hot Strength Compression Test

TIME-TEMPERATURE FIRE CURVE: ASTM E-119

TYPE OF COATING ON FIRE SURFACE: None

TABLE F-1

Specimen Number	Crack Width (inches)					
	0.05		0.10		0.25	
	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress(ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)
1	2.75	0.601	2.55	0.575	2.75	0.060
2	2.55	0.858	2.75	0.610	2.75	0.060
3	2.55	0.798	2.75	0.442	2.95	0.060
4						
Average	2.62	0.752	2.69	0.542	2.82	0.060
Standard Deviation	0.11	0.134	0.11	0.090	0.11	0.000

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The following graphs illustrate the test results provided on page F-2 in TABLE F-1 .

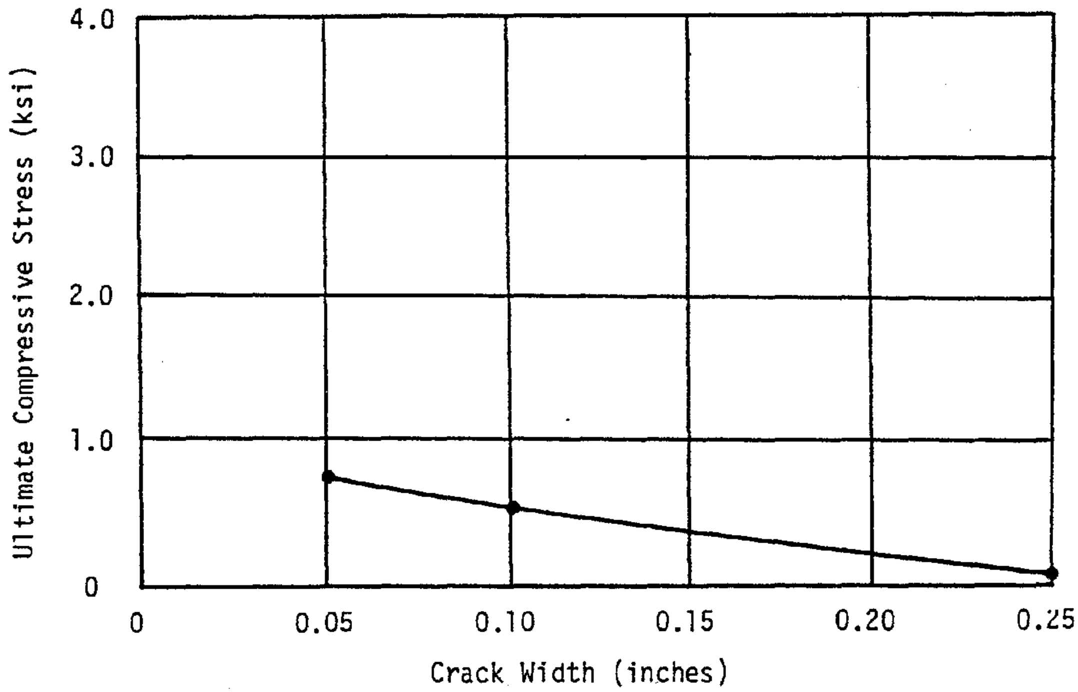

Fig.F-1a : Average Ultimate Compressive Stress as a Function of Crack Width

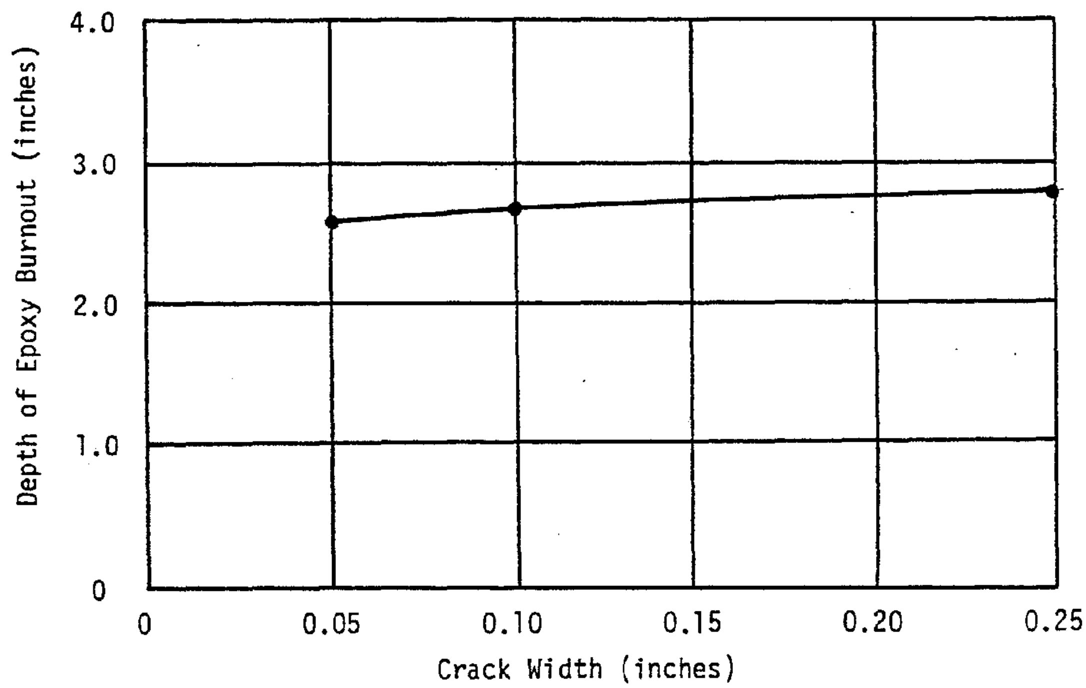

Fig.F-1b : Average Depth of Epoxy Burnout as a Function of Crack Width

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 6 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: High Viscosity (15,000 cps); Structural Grade Epoxy

LOAD CONDITIONS: Hot Strength Compression Test

TIME-TEMPERATURE FIRE CURVE: SDHI

TYPE OF COATING ON FIRE SURFACE: None

TABLE F-2

Specimen Number	Crack Width (inches)					
	0.05		0.10		0.25	
	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress(ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)
1	0.78	0.718	0.78	0.574	1.02	0.417
2	0.70	0.894	0.78	0.893	0.78	0.514
3	0.70	0.952	0.78	0.620	0.90	0.585
4						
Average	0.73	0.855	0.78	0.696	0.90	0.505
Standard Deviation	0.04	0.122	0.00	0.172	0.11	0.084

STRUCTURES LABORATORY AT CALIFORNIA STATE UNIVERSITY, LONG BEACH

The following graphs illustrate the test results provided on page F-4 in TABLE F-2.

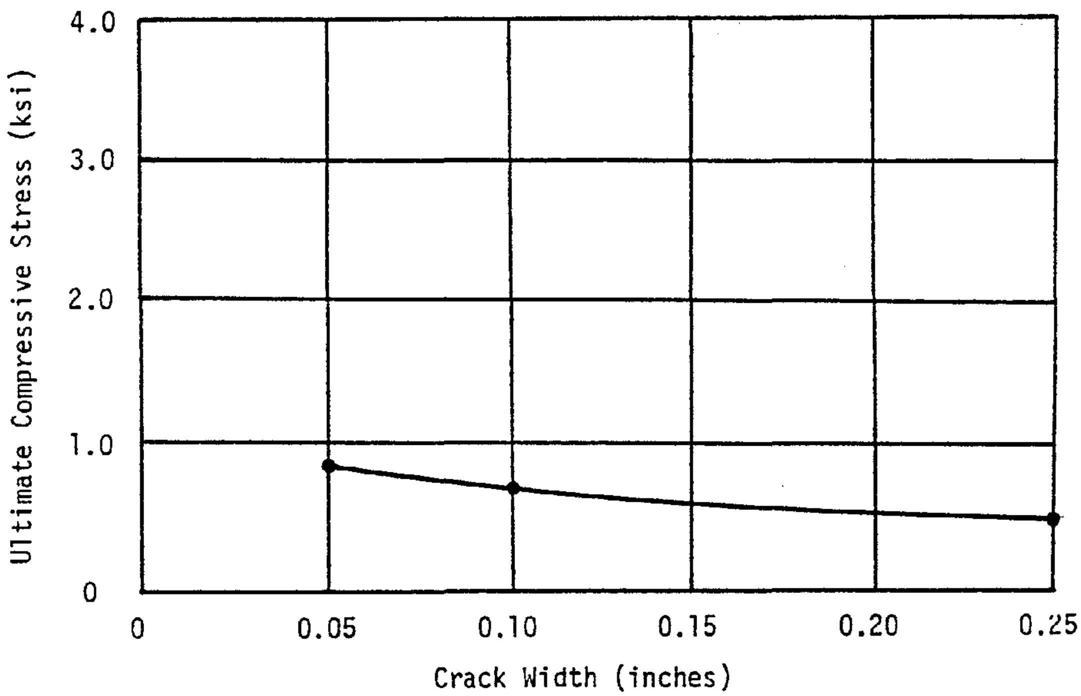

Fig. F-2a : Average Ultimate Compressive Stress as a Function of Crack Width

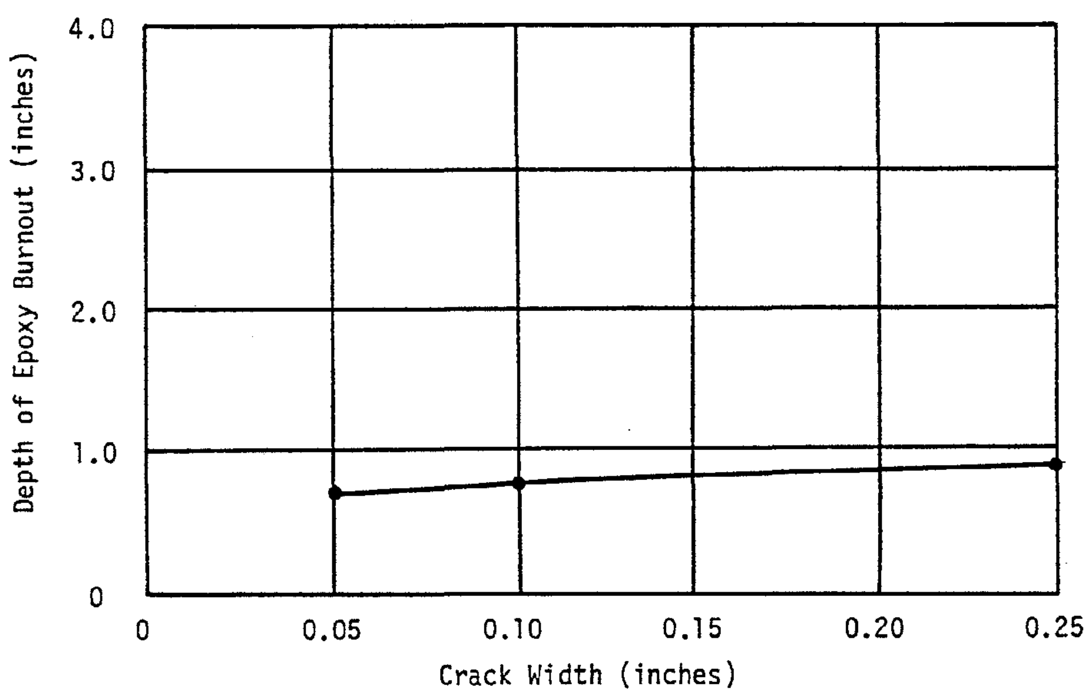

Fig.F-2b : Average Depth of Epoxy Burnout as a Function of Crack Width

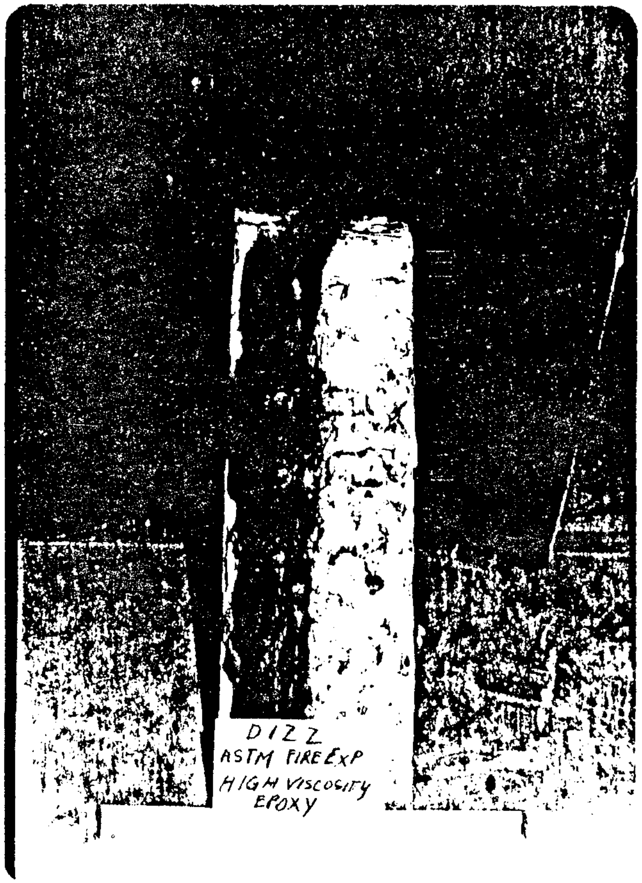

Fig. F-3a : Failure pattern for 6 in. Thick Specimen, High Viscosity Epoxy and ASTM E-119 Fire Exposure.

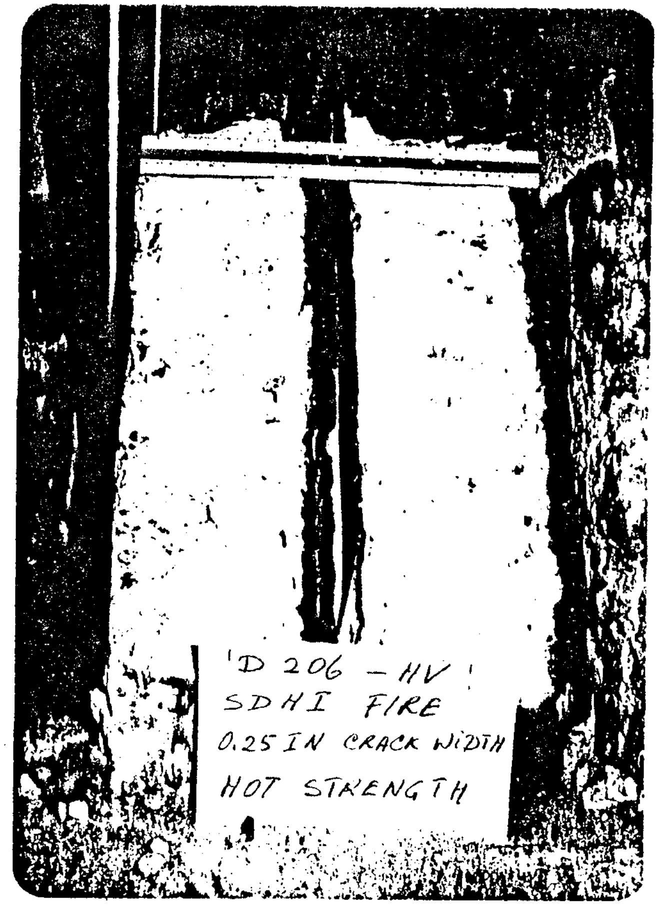

Fig. F-3b : Failure pattern for 6 in. Thick Specimen, High Viscosity Epoxy and SDHI Fire Exposure.

CHAPTER G: ASTM E119 HOT STRENGTH COMPRESSION TEST RESULTS (PLASTER FIRE SURFACE COATING: LOW VISCOSITY EPOXY)

SEC. G.1: TEST PROCEDURE AND TEST PARAMETERS

This chapter provides a complete summary of test results for specimens whose dimensions and load application are described in Chapter A. The epoxy used to repair all cracks consisted of low viscosity type epoxies which have been described in Chapter A. All test specimens considered in this Chapter G have been exposed to the standard 2-hour ASTM E119 fire exposure for walls. Primary test parameters studied in this chapter include crack widths of 0.05 in., 0.10 in., and 0.25 in., wall thickness of 6 in., and plaster coating on fire exposed surface of 1 in. and 3/8 in. thickness. The plaster was applied to the specimens as described in Chapter A. All specimens have been subjected to the ultimate compression loads immediately after the 2-hour fire exposure.

SEC. G.2: SUMMARY OF TEST RESULTS

Table G-1 provides the test data and Fig. G-1 provides the corresponding graphical summary for specimens that have had a 1 in. plaster coating applied to the fire exposed surface. Table G-2 provides the test data and Fig. G-2 provides the corresponding graphical summary for specimens that have had a 3/8 in. plaster coating applied to the fire exposed surface. Fig. G-5 provides a pictorial view of the epoxy burnout and failure pattern. Comparison of results in this Chapter with corresponding unplastered test results in Chapter B indicates that 1 in. thick plaster coating is extremely effective in reducing depth of epoxy burnout but is not effective in increasing ultimate "hot" stress as illustrated in Fig. G-3. The lower effectiveness of 3/8 in. thick plaster coating is also illustrated in Fig. G-4. The effectiveness of both the 1 in. and the 3/8 in. thick plaster coatings in reducing the depth of epoxy burnout indicates that "residual strengths" are increased substantially by the application of both the 1 in. and the 3/8 in. thick plaster coatings.

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 6 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: Low Viscosity (400 cps); Structural Grade Epoxy.

LOAD CONDITIONS: **Hot Strength Compression Test**

TIME-TEMPERATURE FIRE CURVE: ASTM E-119

TYPE OF COATING ON FIRE SURFACE: 1 inch Plaster Coating

TABLE G-1

Specimen Number	Crack Width (inches)					
	0.05		0.10		0.25	
	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)
1	0.78	0.489	0.78	0.387	0.78	0.244
2	0.70	0.423	0.78	0.489	0.78	0.351
3	0.70	0.762	0.39	0.590	0.78	0.244
4	0.59	0.798				
Average	0.69	0.618	0.65	0.489	0.78	0.280
Standard Deviation	0.08	0.190	0.22	0.102	0.00	0.062

STRUCTURES LABORATORY AT CALIFORNIA STATE UNIVERSITY, LONG BEACH

The following graphs illustrate the test results provided on page G-2 in TABLE G-1 .

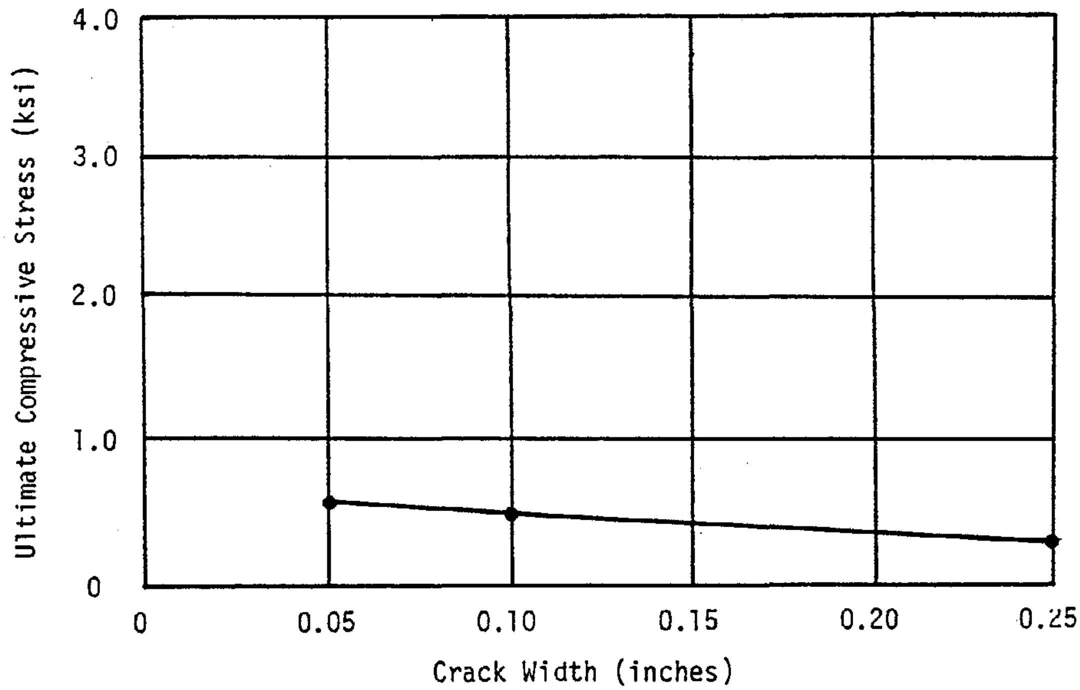

Fig.G-1a : Average Ultimate Compressive Stress as a Function of Crack Width

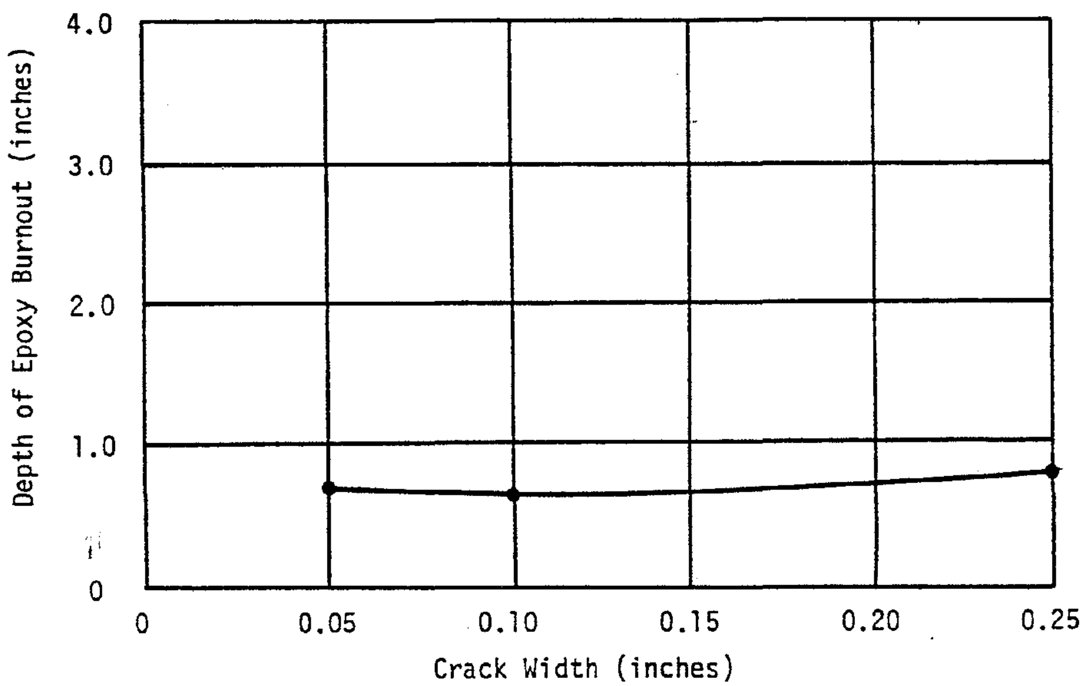

Fig.G-1b : Average Depth of Epoxy Burnout as a Function of Crack Width

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 6 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: Low Viscosity (400 cps); Structural Grade Epoxy.

LOAD CONDITIONS: Hot Strength Compression Test

TIME-TEMPERATURE FIRE CURVE: ASTM E-119

TYPE OF COATING ON FIRE SURFACE: 3/8 inch Plaster Coating

TABLE G-2

Specimen Number	Crack Width (inches)					
	0.05		0.10		0.25	
	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)
1	1.77	0.507	1.57	0.439	1.96	0.146
2	1.57	0.762	2.16	0.455	1.18	0.146
3	1.37	0.619	1.81	0.524	1.57	0.119
4						
Average	1.57	0.629	1.85	0.473	1.57	0.137
Standard Deviation	0.19	0.128	0.29	0.045	0.39	0.016

The following graphs illustrate the test results provided on page G-4 in TABLE G-2 :

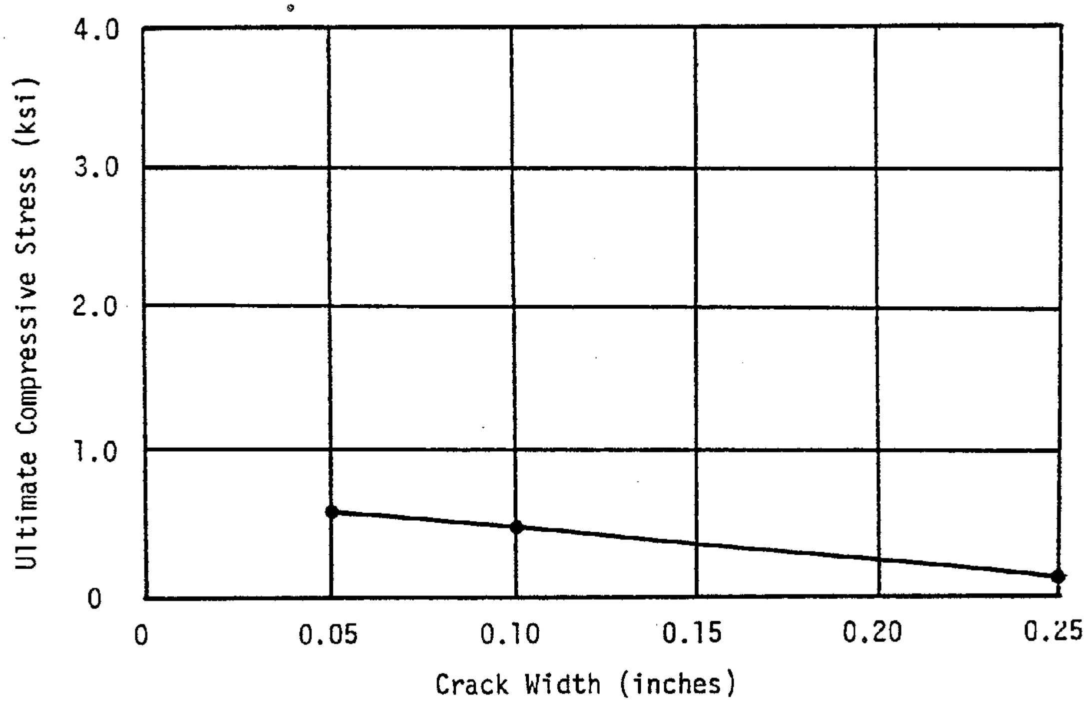

Fig.G-2a : Average Ultimate Compressive Stress as a Function of Crack Width

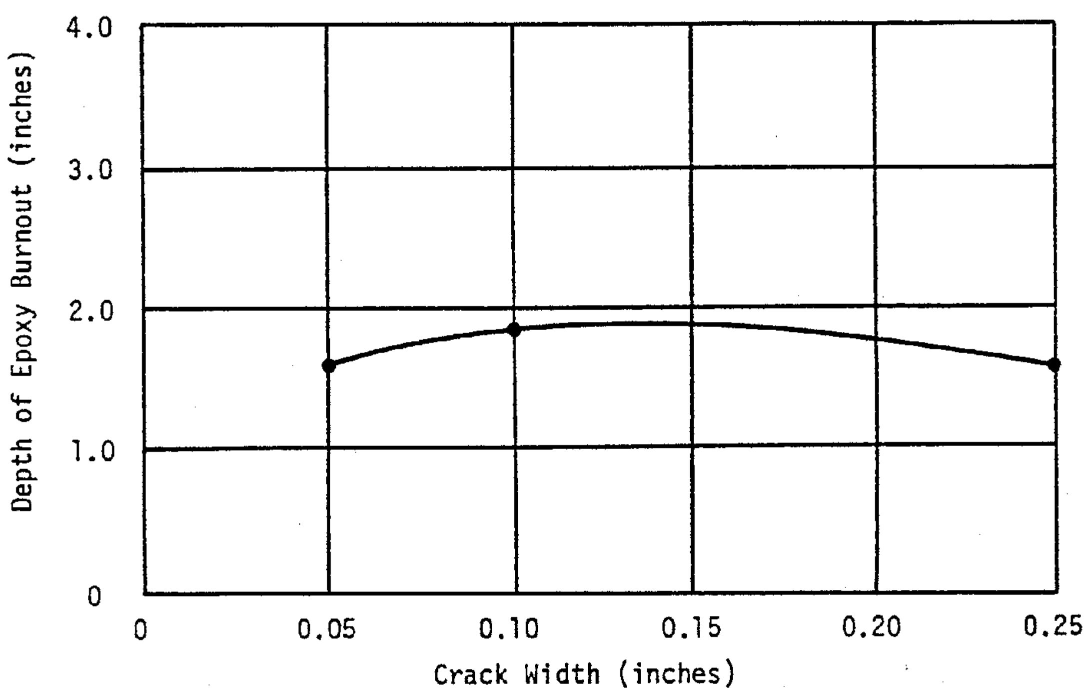

Fig.G-2b : Average Depth of Epoxy Burnout as a Function of Crack Width

The following graphs illustrate the test results provided on page B-2,G-2 in TABLE B-1,G-

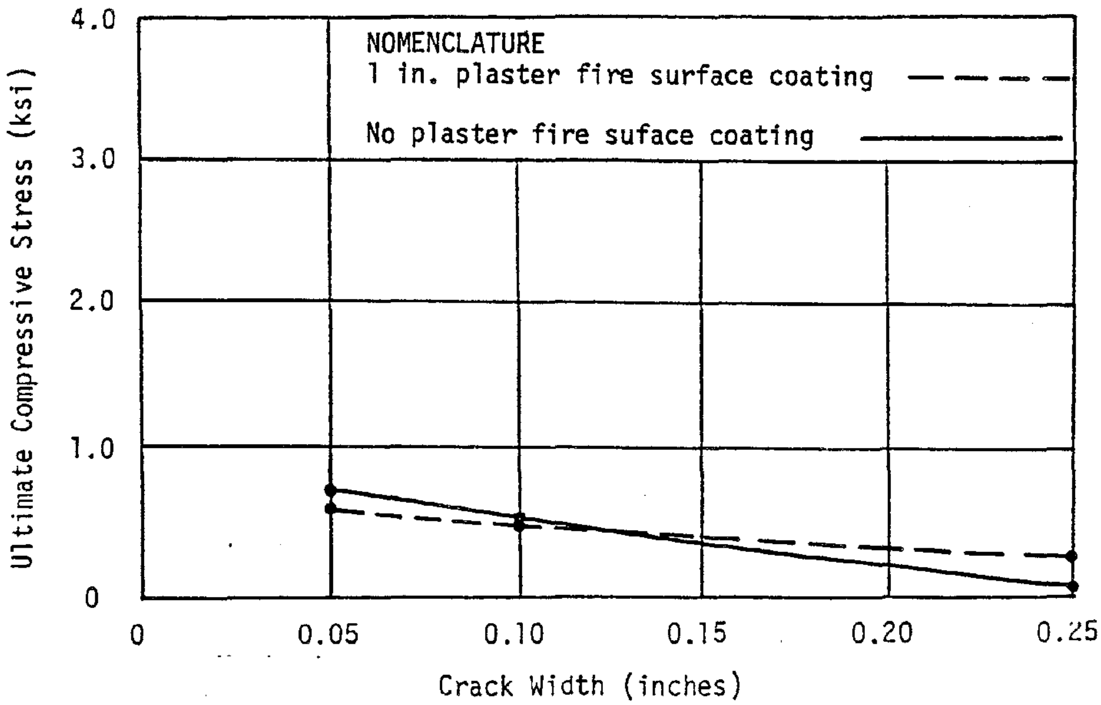

Fig. G-3a: Average Ultimate Compressive Stress as a Function of Crack Width

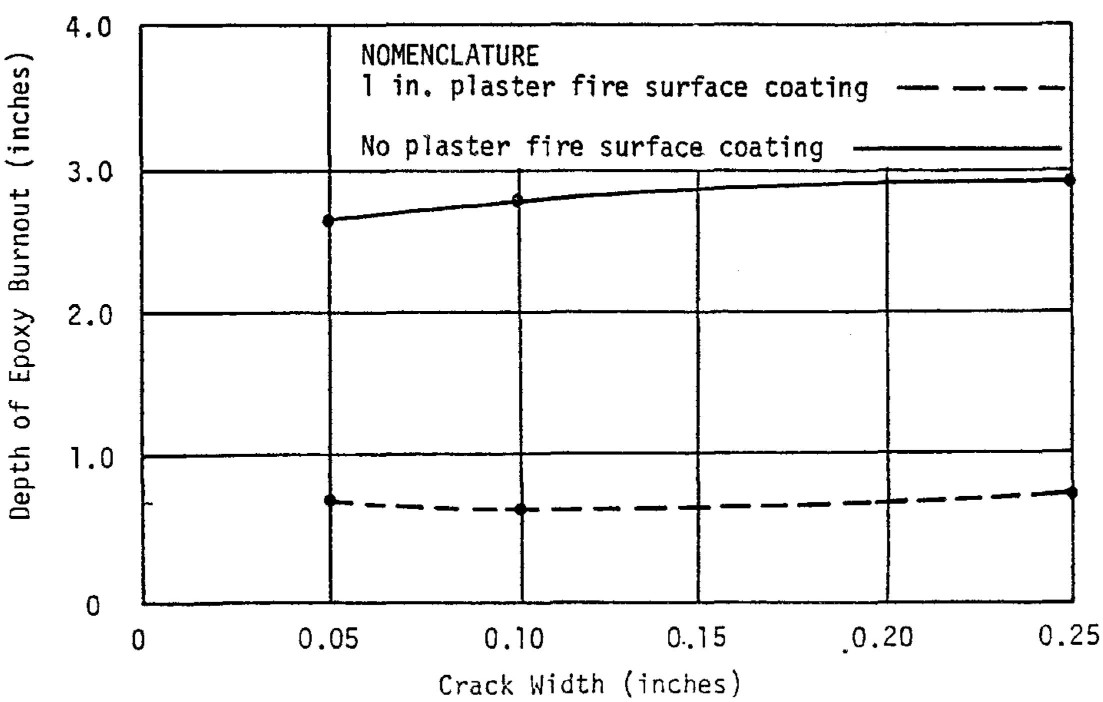

Fig. G-3b: Average Depth of Epoxy Burnout as a Function of Crack Width

The following graphs illustrate the test results provided on page B-2,G-4 in TABLE B-1,G-2

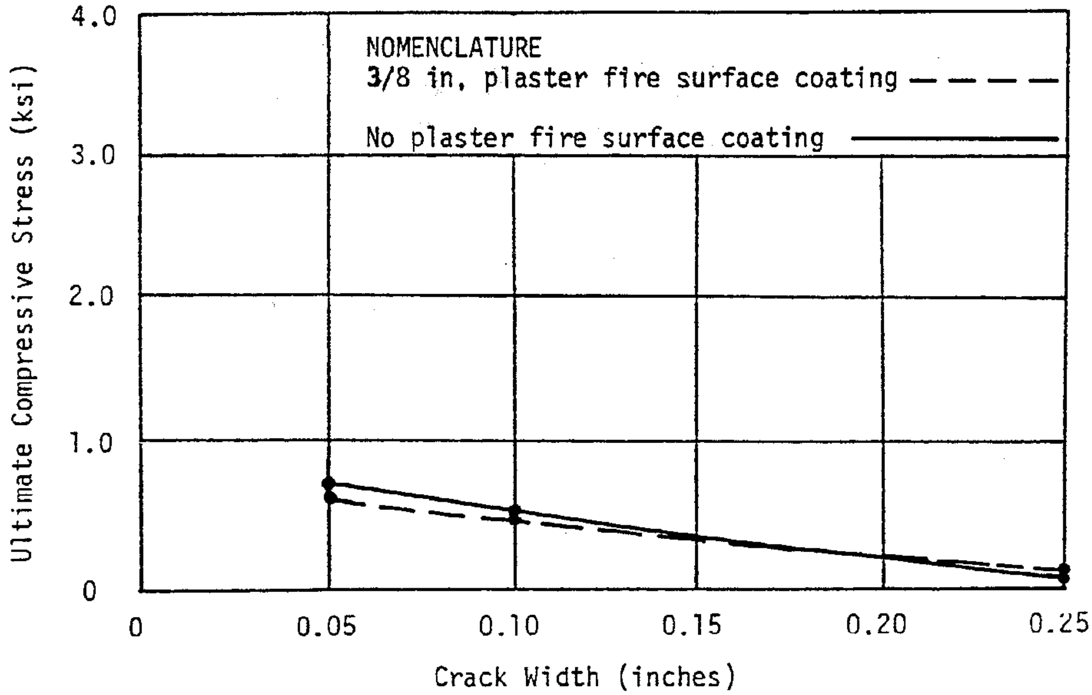

Fig. G-4a : Average Ultimate Compressive Stress as a Function of Crack Width

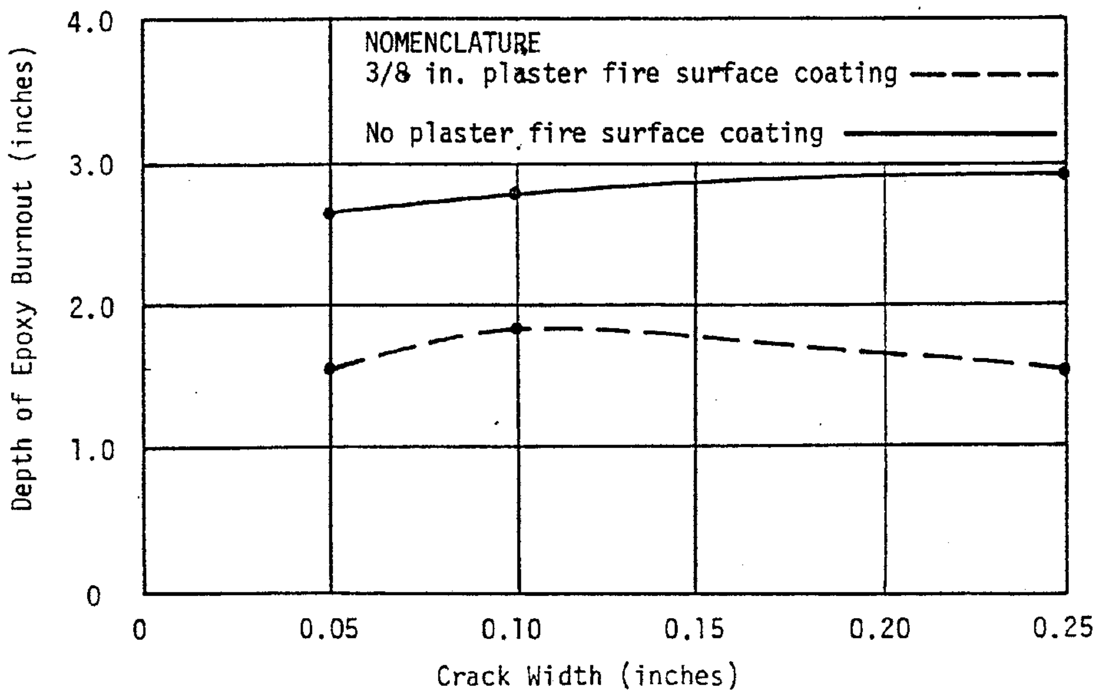

Fig. G-4b : Average Depth of Epoxy Burnout as a Function of Crack Width

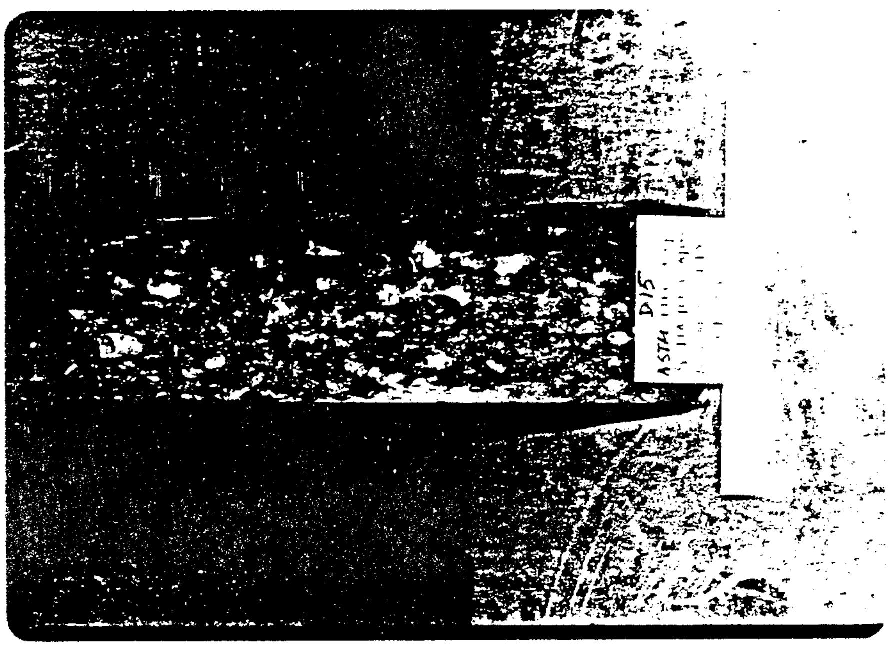

Fig. G-5a : Failure pattern for 6 in. thick Specimen with 3/8 in. Fire surface Coating.

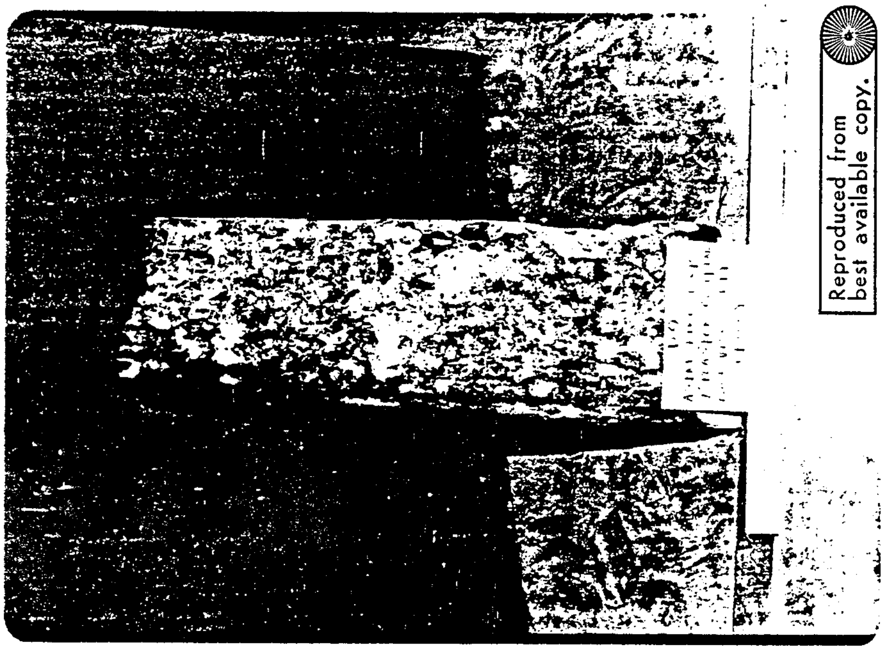

Fig. G-5b : Failure pattern for 6 in. thick Specimen with 1 in. Fire Surface Coating.

CHAPTER H: SDHI HOT STRENGTH COMPRESSION TEST RESULTS (PLASTER FIRE SURFACE COATING; LOW VISCOSITY EPOXY)

SEC. H.1: TEST PROCEDURE AND TEST PARAMETERS

This chapter provides a complete summary of test results for specimens whose dimensions and load application are described in Chapter A. The epoxy used to repair all cracks consisted of low viscosity type epoxies which have been described in Chapter A. All test specimens considered in this Chapter H have been exposed to the standard SDHI fire exposure for walls. Primary test parameters studied in this chapter include crack widths of 0.05 in., 0.10 in., and 0.25 in., wall thickness of 6 in., and plaster coating on fire exposed surface of 1 in. and 3/8 in. thickness. The plaster was applied to the specimens as described in Chapter A. All specimens have been subjected to the ultimate compression loads immediately after the fire exposure.

SEC. H.2: SUMMARY OF TEST RESULTS

Table H-1 provides the test data and Fig. H-1 provides the corresponding graphical summary for specimens that have had a 1 in. plaster coating applied to the fire exposed surface. Table H-2 provides the test data and Fig. H-2 provides the corresponding graphical summary for specimens that have had a 3/8 in. plaster coating applied to the fire exposed surface. Shear failure through the concrete was the most common type of failure pattern. Comparison of results in this Chapter with corresponding unplastered test results in Chapter C indicates that 1 in. thick plaster coating is extremely effective in reducing depth of epoxy burnout and increasing ultimate compressive stress as illustrated in Fig. H-3. The lower effectiveness of 3/8 in. thick plaster coating is also illustrated in Fig. H-4. Ultimate compressive stress is a funciton of crack width due to the development of higher frictional forces between concrete surfaces in the case of smaller crack widths. Depth of epoxy burnout for all crack widths and for both the 3/8 in. and the 1 in. thick plaster coatings was zero for all test specimens.

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 6 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: Low Viscosity (400 cps); Structural Grade Epoxy.

LOAD CONDITIONS: Hot Strength Compression Test

TIME-TEMPERATURE FIRE CURVE: SDHI

TYPE OF COATING ON FIRE SURFACE: 1 inch Plaster Coating

TABLE H-1

Specimen Number	Crack Width (inches)					
	0.05		0.10		0.25	
	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)
1	0	2.167	0	2.417	0	1.280
2	0	2.679	0	1.440	0	1.563
3						
4						
Average	0	2.423	0	1.929	0	1.422
Standard Deviation	0	0.362	0	0.691	0	0.200

STRUCTURES LABORATORY AT CALIFORNIA STATE UNIVERSITY, LONG BEACH

The following graphs illustrate the test results provided on page H-2 in TABLE H-1.

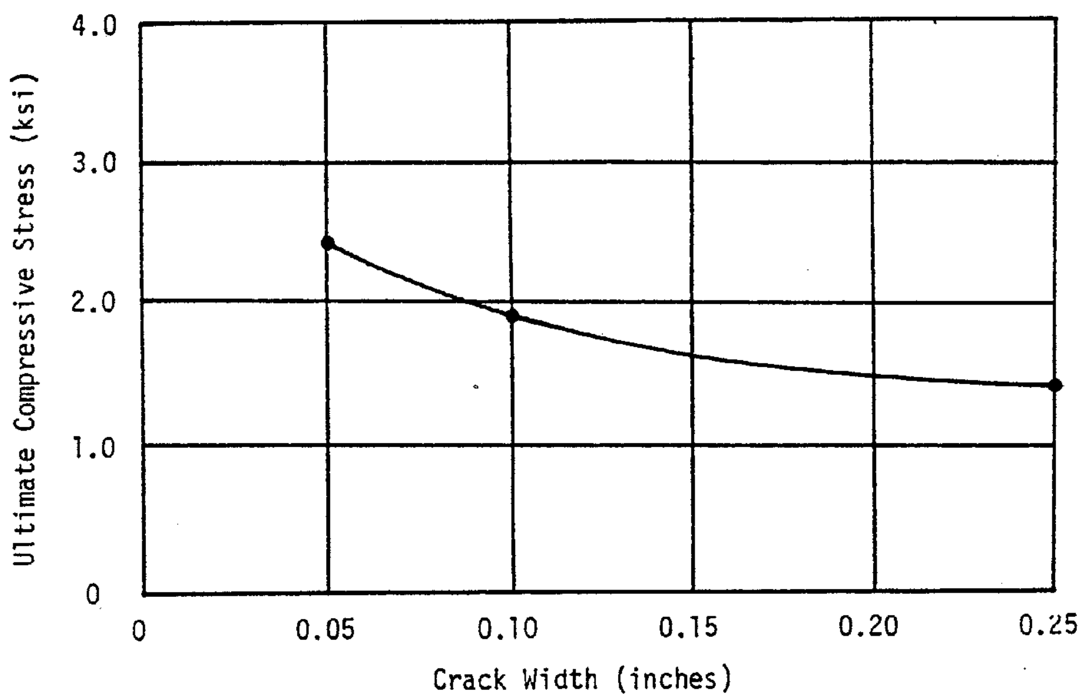

Fig. H-1a : Average Ultimate Compressive Stress as a Function of Crack Width

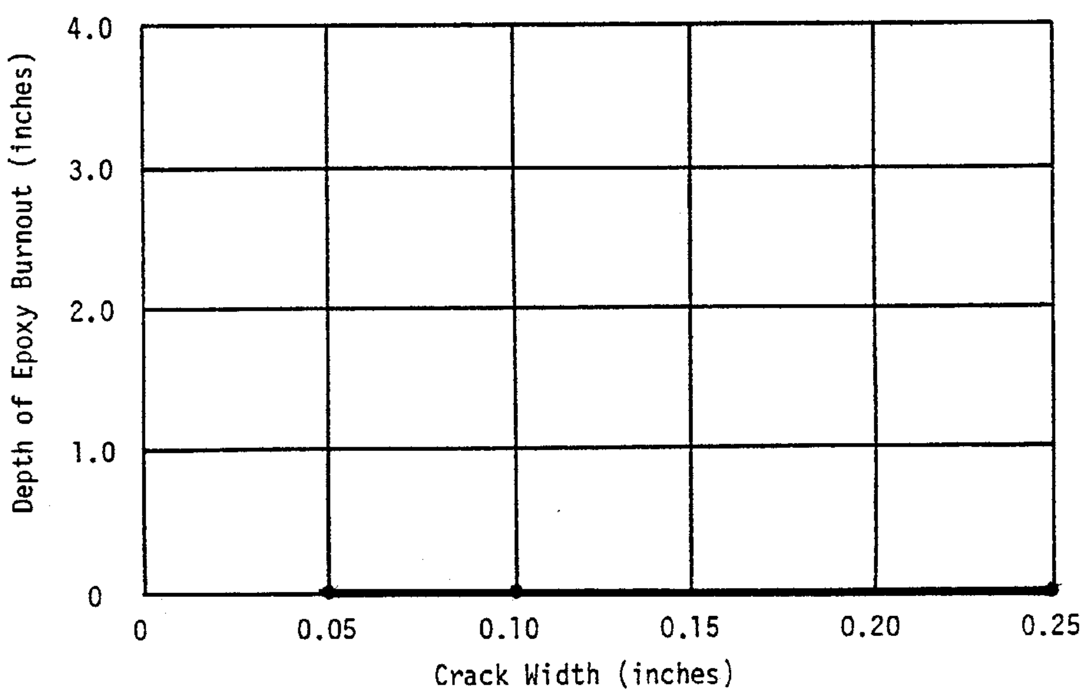

Fig. H-1b : Average Depth of Epoxy Burnout as a Function of Crack Width

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 6 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: Low Viscosity (400 cps); Structural Grade Epoxy.

LOAD CONDITIONS: Hot Strength Compression Test

TIME-TEMPERATURE FIRE CURVE: SDHI

TYPE OF COATING ON FIRE SURFACE: 3/8 inch Plaster Coating

TABLE H-2

Specimen Number	Crack Width (inches)					
	0.05		0.10		0.25	
	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress(ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)
1	0.00	1.462	0.00	0.625	0.00	0.599
2	0.00	1.170	0.00	0.814	0.00	0.500
3						
4						
Average	0.00	1.316	0.00	0.720	0.00	0.550
Standard Deviation	0.00	0.206	0.00	0.134	0.00	0.070

STRUCTURES LABORATORY AT CALIFORNIA STATE UNIVERSITY, LONG BEACH

The following graphs illustrate the test results provided on page H-4 in TABLE H-2 .

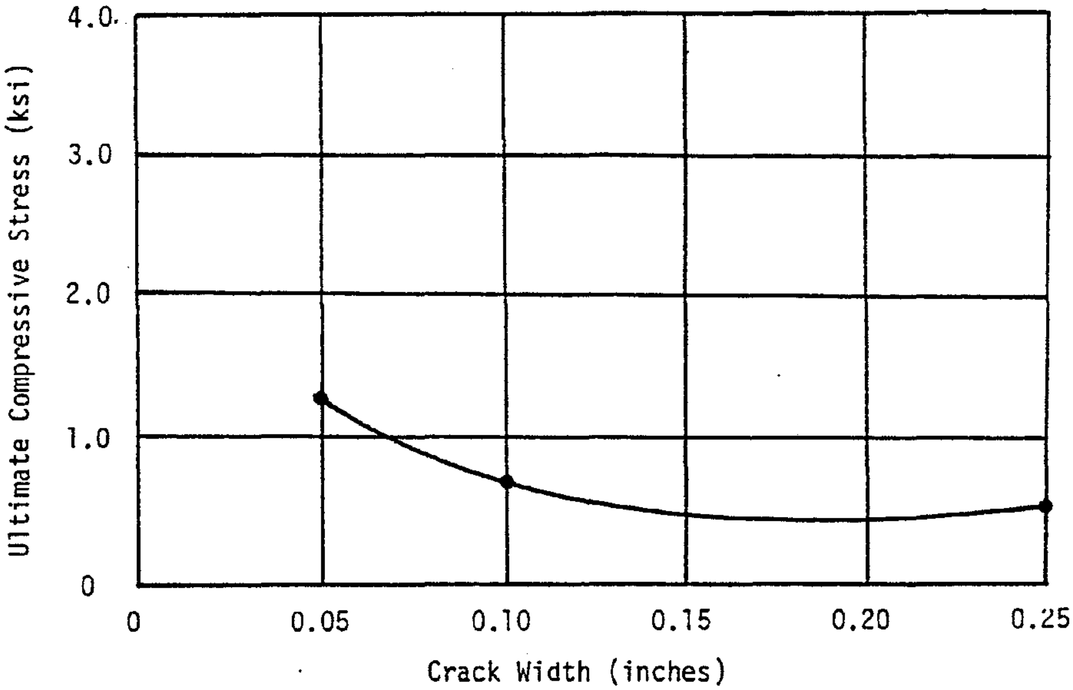

Fig. H-2a : Average Ultimate Compressive Stress as a Function of Crack Width

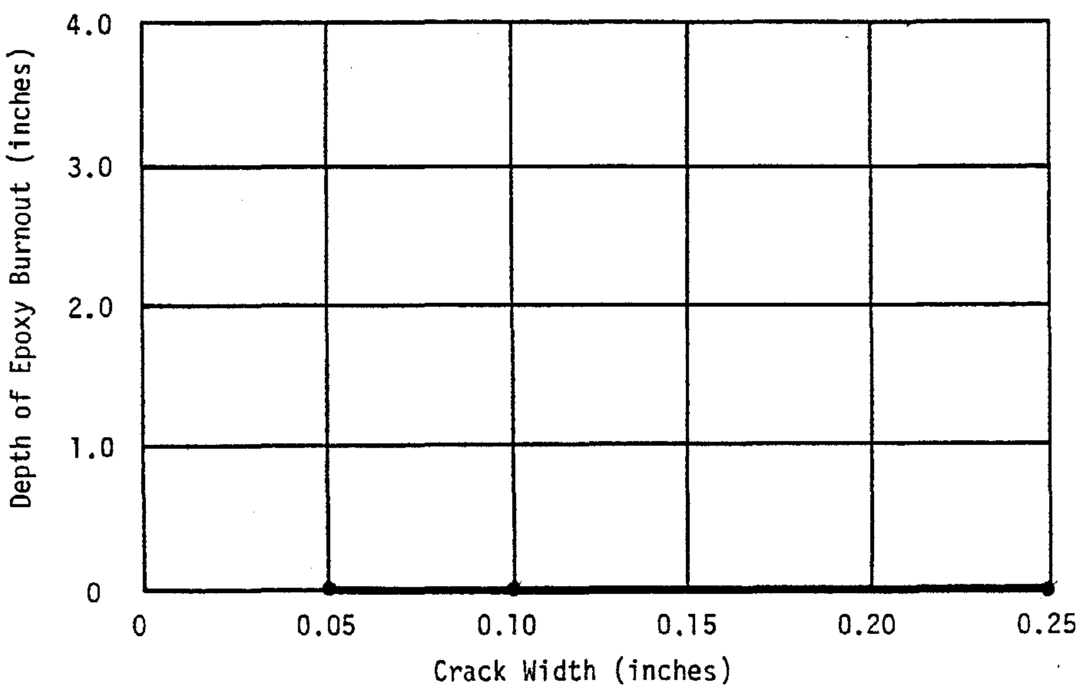

Fig.H-2b : Average Depth of Epoxy Burnout as a Function of Crack Width

STRUCTURES LABORATORY AT CALIFORNIA STATE UNIVERSITY, LONG BEACH

The following graphs illustrate the test results provided on page C-2,H-2 in TABLE C-1,H-1

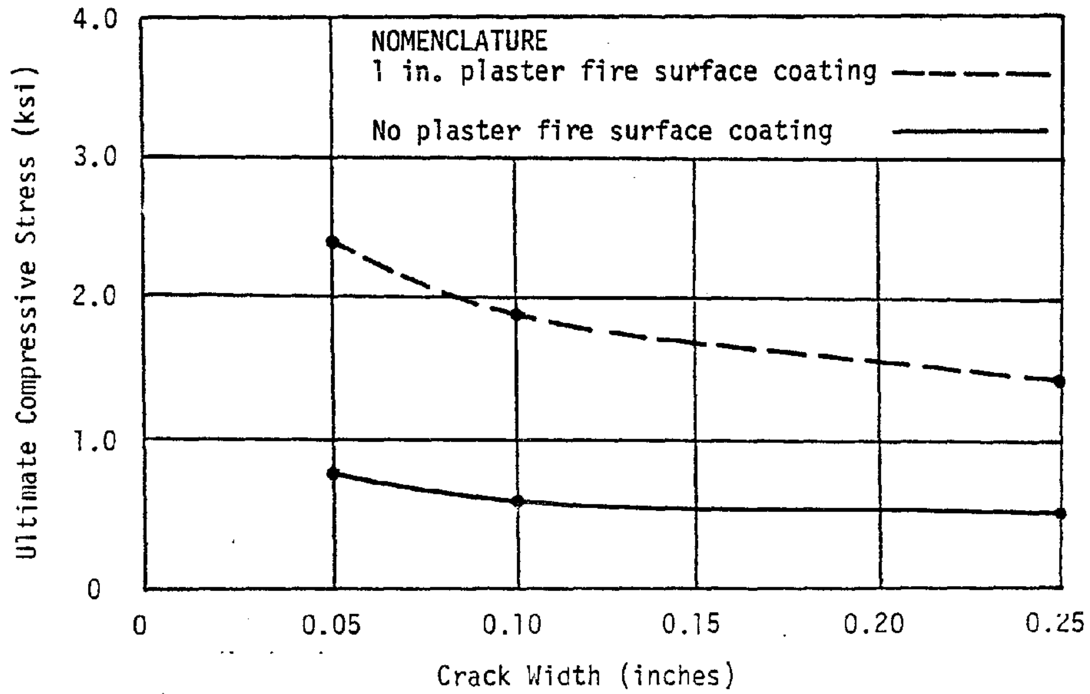

Fig. H-3a: Average Ultimate Compressive Stress as a Function of Crack Width

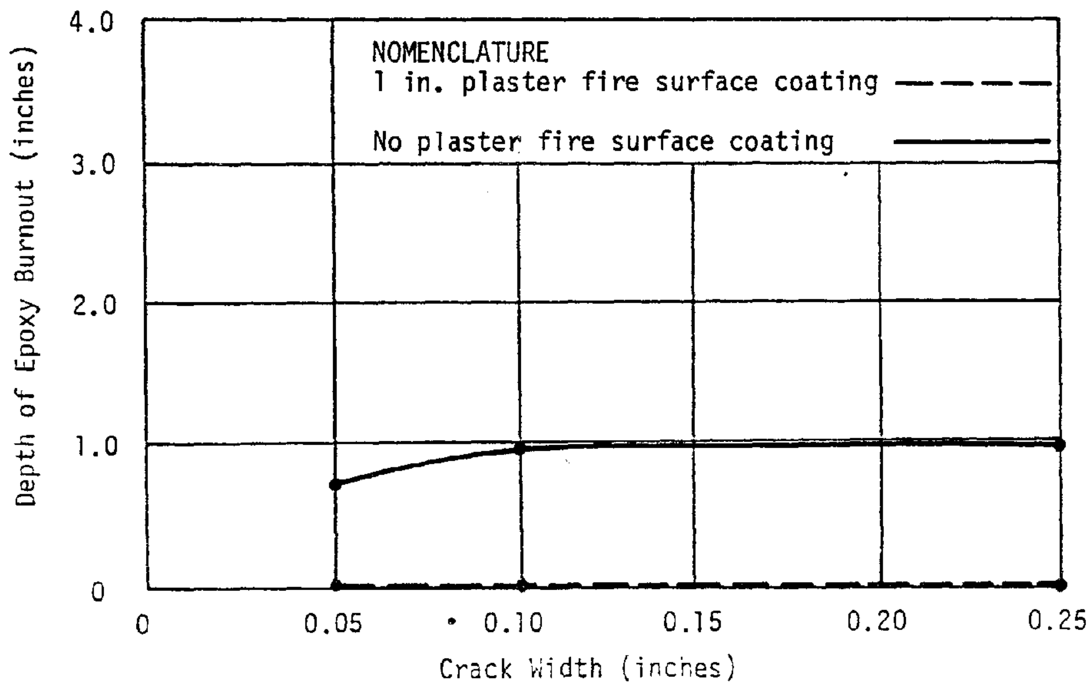

Fig. H-3b: Average Depth of Epoxy Burnout as a Function of Crack Width

The following graphs illustrate the test results provided on page C-2,H-4 in TABLE C-1,H-2

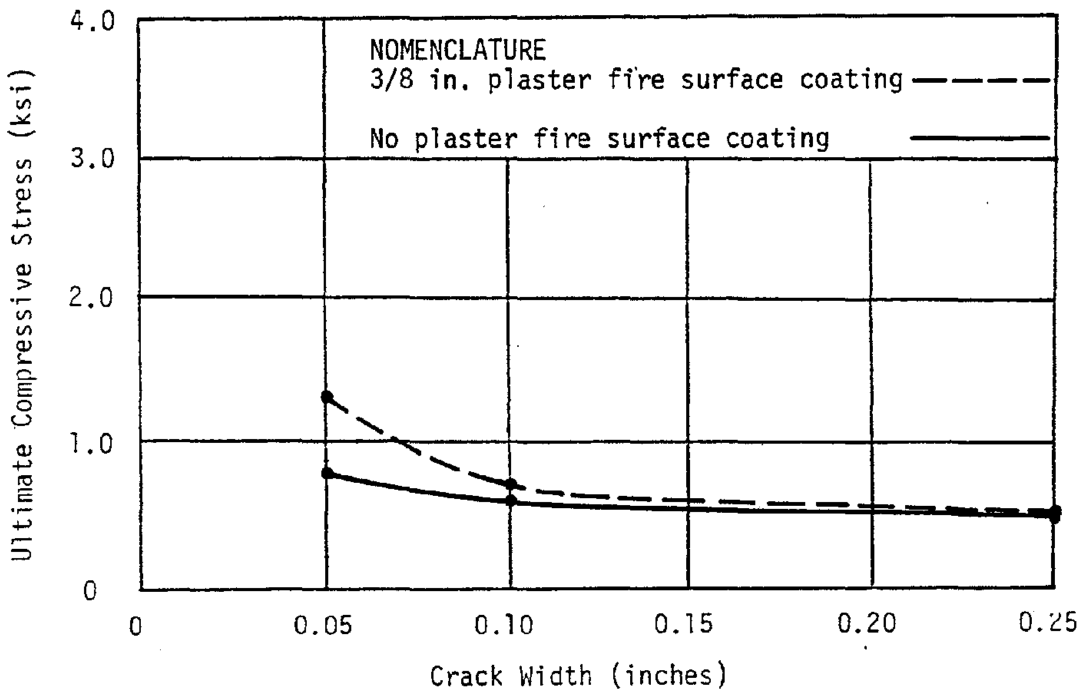

Fig. H-4a: Average Ultimate Compressive Stress as a Function of Crack Width

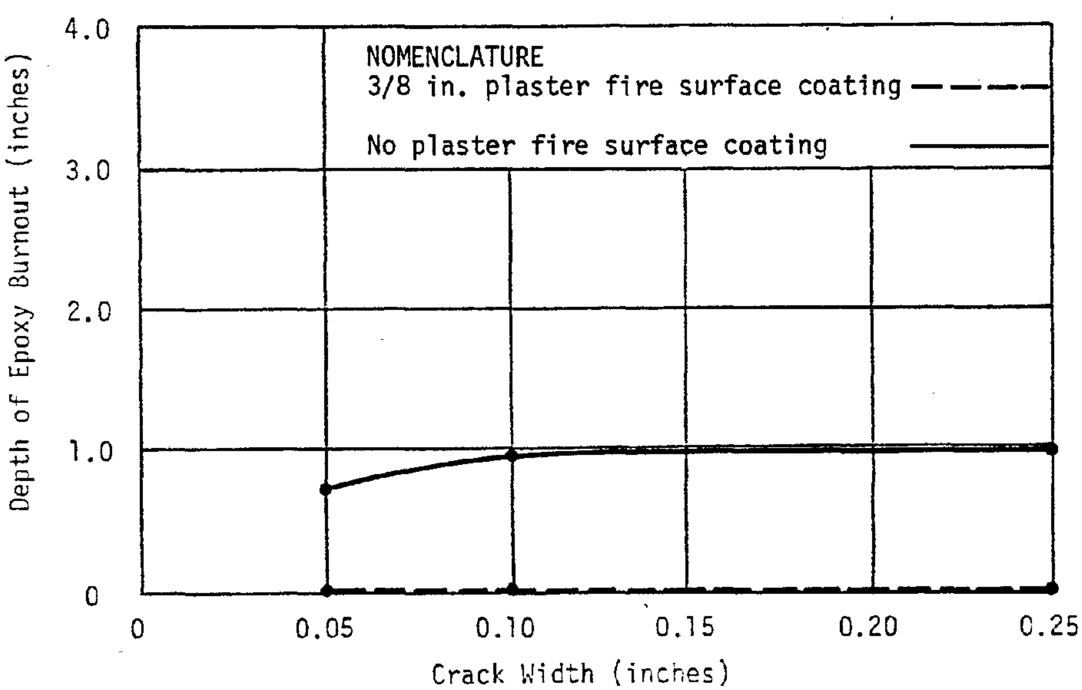

Fig. H-4b: Average Depth of Epoxy Burnout as a Function of Crack Width

CHAPTER I. ASTM E119 HOT STRENGTH COMPRESSION TEST RESULTS (ORGANIC FIRE SURFACE COATING; LOW VISCOSITY EPOXY)

SEC. I.1: TEST PROCEDURE AND TEST PARAMETERS

This chapter provides a complete summary of test results for specimens whose dimensions and load application are described in Chapter A. The epoxy used to repair all cracks consisted of low viscosity type epoxies which have been described in Chapter A. All test specimens considered in this Chapter I have been exposed to the standard 2-hour ASTM E119 fire exposure for walls. Primary test parameters studied in this chapter include crack width of 0.10 in., wall thickness of 6 in. and organic fire retardent coatings were applied to the specimens as described in Chapter A. All specimens have been subjected to the ultimate compression loads immediately after the 2-hour fire exposure.

SEC. I.2: SUMMARY OF TEST RESULTS

Table I-1 provides the test data for each specimen including the ultimate compression strength and the depth of epoxy burnout. Comparison of these test results with the uncoated test results in Chapter B indicates that thin organic surface coatings, including both the fire retardant intumescent paints and fire resistant epoxy foams, are not effective fire surface coatings. Fig. I-1 provides pictorial view of the failure pattern which is identical to that for specimens in Chapter B where fire surface coating were not provided.

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 6 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: Low Viscosity (400 cps); Structural Grade Epoxy

LOAD CONDITIONS: Hot Strength Compression Test

TIME-TEMPERATURE FIRE CURVE: ASTM E-119

TYPE OF COATING ON FIRE SURFACE: Fire Resistant Epoxy Foams, Intumescent paints

CRACK WIDTH : 0.10 in. for all specimens

TABLE I-1

Specimen Number	Thickness of Coating (inches)			
	0.050		0.100	
	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)
1	2.67	0.456	2.55	0.625
2	2.95	0.244	2.75	0.423
3	2.75	0.536	2.75	0.244
4				
Average	2.79	0.412	2.69	0.431
Standard Deviation	0.14	0.151	0.11	0.191

Fig. I-1 : Failure pattern for 6 in. Thick Specimen with Organic Fire Surface Coating.

CHAPTER J: ASTM E119 AND SDHI RE-INJECTED TEST RESULTS (NO FIRE SURFACE COATING; LOW VISCOSITY EPOXY)

SEC. J.1: TEST PROCEDURE AND TEST PARAMETERS

This chapter provides a complete summary of test results for specimens whose dimensions and load application are described in Chapter A. The epoxy used to repair all cracks consisted of low viscosity type epoxies which have been described in Chapter A. All test specimens considered in this Chapter J have been exposed to the standard two-hour ASTM E119 or SDHI fire exposure for walls. Primary test parameters studied in this chapter include crack widths of 0.05 in., 0.10 in. and 0.25 in. and wall thickness of 6 in. Each specimen was subjected to the prescribed fire exposure, cooled for seven days under laboratory conditions, the burnout crack cleaned with pressurized air and a wire brush and subsequently repaired with re-injected low viscosity epoxy adhesives and mortar mix. The repaired specimens were cured for 28 days and tested in compression under laboratory conditions as all other specimens in this report.

SEC. J.2: SUMMARY OF TEST RESULTS

Tables J-1 and J-2 provide the test data for each specimen including the ultimate compression strength and the initial depth of epoxy burnout. Figs. J-1 and J-2 provide the graphical summary of average test results including average ultimate compressive stress and initial depth of epoxy burnout as a function of crack width. The initial depth of epoxy burnout was determined after the specimen had been cooled but prior to re-injection of epoxy adhesives. Results in both Figs. J-1 and J-2 indicate that the ultimate compressive stress of re-injected specimens was not significantly affected by crack width. However, the ultimate compressive stress test results for ASTM E119 fire exposure as given in Fig. J-1 are significantly lower than the SDHI test results in Fig. J-2. Failure pattern for all specimens in this chapter consisted of shear failure in the concrete. Based on the observations of the failed specimens, it appears that re-injection procedures as utilized for these specimens, were extremely effective in the repair of epoxy repaired shear walls which had been subjected to fire exposure.

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 6 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: Low Viscosity (400 cps); Structural Grade Epoxy

LOAD CONDITIONS: Compression Test after Re-Injection

TIME-TEMPERATURE FIRE CURVE: ASTM E-119

TYPE OF COATING ON FIRE SURFACE: None

TABLE J-1

Specimen Number	Crack Width (inches)					
	0.05		0.10		0.25	
	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress(ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)
1	2.80	3.014	2.85	2.821	3.0	2.595
2	2.50	2.304	3.20	1.804	3.0	2.143
3						
4						
Average	2.65	2.659	3.02	2.313	3.0	2.369
Standard Deviation	0.21	0.502	0.24	0.719	0.0	0.320

The following graphs illustrate the test results provided on page J-3 in TABLE J-1.

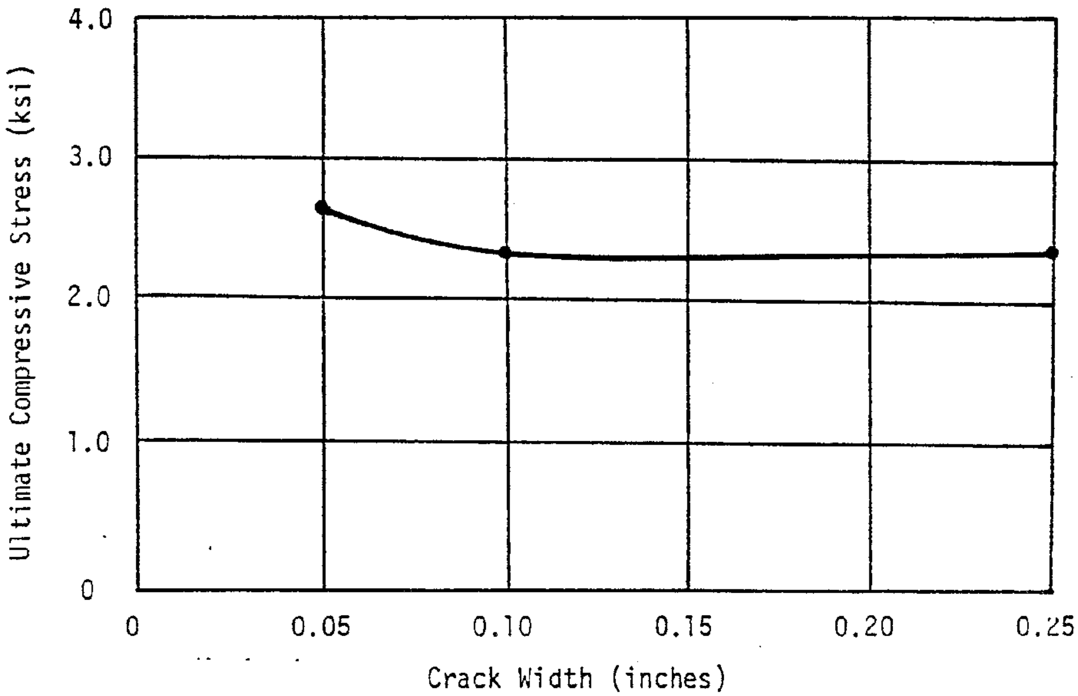

Fig. J-1a: Average Ultimate Compressive Stress as a Function of Crack Width

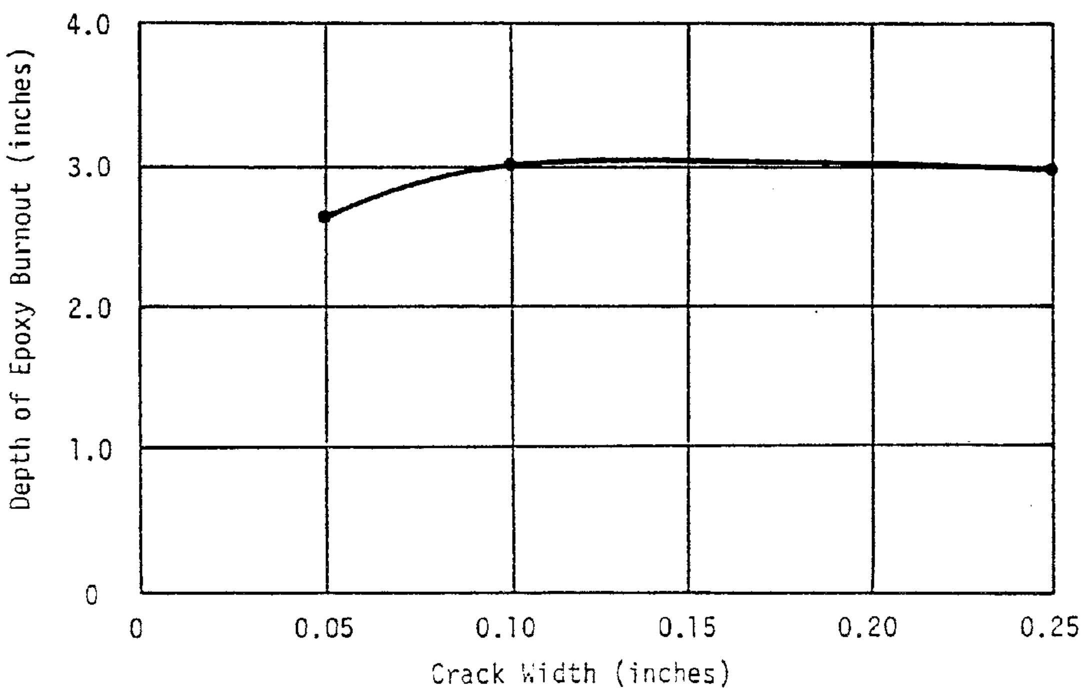

Fig. J-1b: Average Depth of Epoxy Burnout as a Function of Crack Width

SUMMARY OF EXPERIMENTAL TEST RESULTS

The experimental test results presented in the table below correspond to the following test conditions for the epoxy repaired concrete shear wall specimens.

SPECIMEN SIZE: Width = 14 in.; Height = 18 in.; Thickness = 6 in.

CONCRETE TYPE: Normal Weight; Unreinforced; 4.0 ksi Compressive Strength

EPOXY TYPE: Low Viscosity (400 cps); Structural Grade Epoxy.

LOAD CONDITIONS: Compression Test after Re-Injection

TIME-TEMPERATURE FIRE CURVE: SDHI

TYPE OF COATING ON FIRE SURFACE: None

TABLE J-2

Specimen Number	Crack Width (inches)					
	0.05		0.10		0.25	
	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)	Burnout (inches)	Ultimate Compressive Stress (ksi)
1	0.85	3.319	1.42	3.571	1.35	3.333
2	0.79	3.571	1.05	3.512	0.95	3.260
3						
4						
Average	0.82	3.445	1.24	3.542	1.15	3.297
Standard Deviation	0.04	0.178	0.26	0.042	0.28	0.052

STRUCTURES LABORATORY AT CALIFORNIA STATE UNIVERSITY, LONG BEACH

The following graphs illustrate the test results provided on page J-5 in TABLE J-2 .

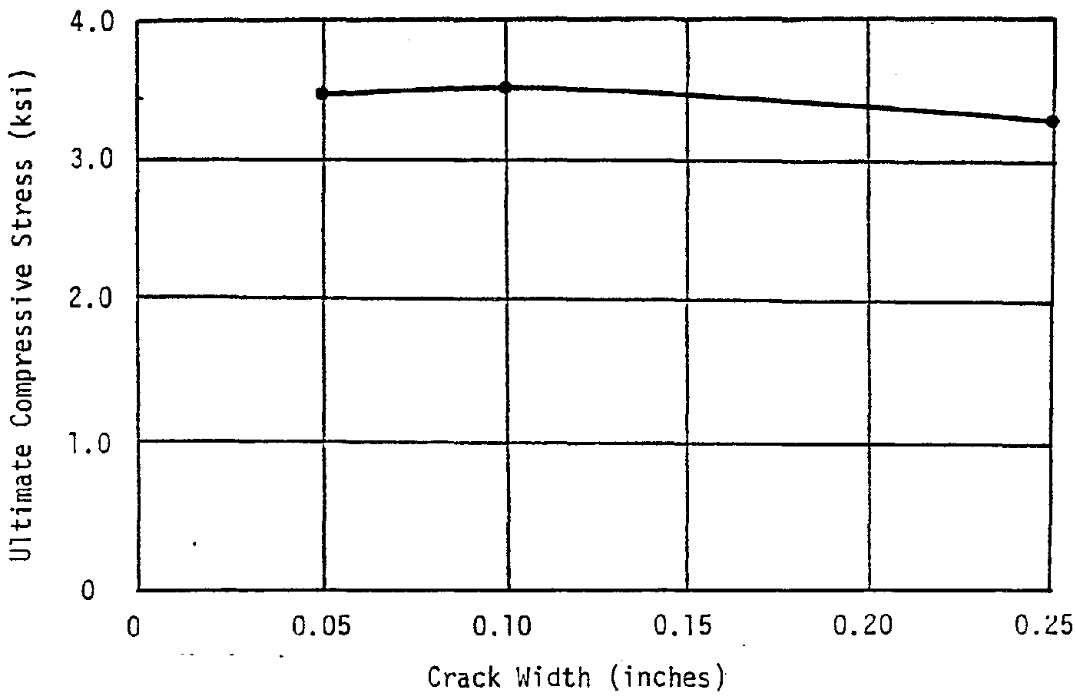

Fig. J-2a : Average Ultimate Compressive Stress as a Function of Crack Width

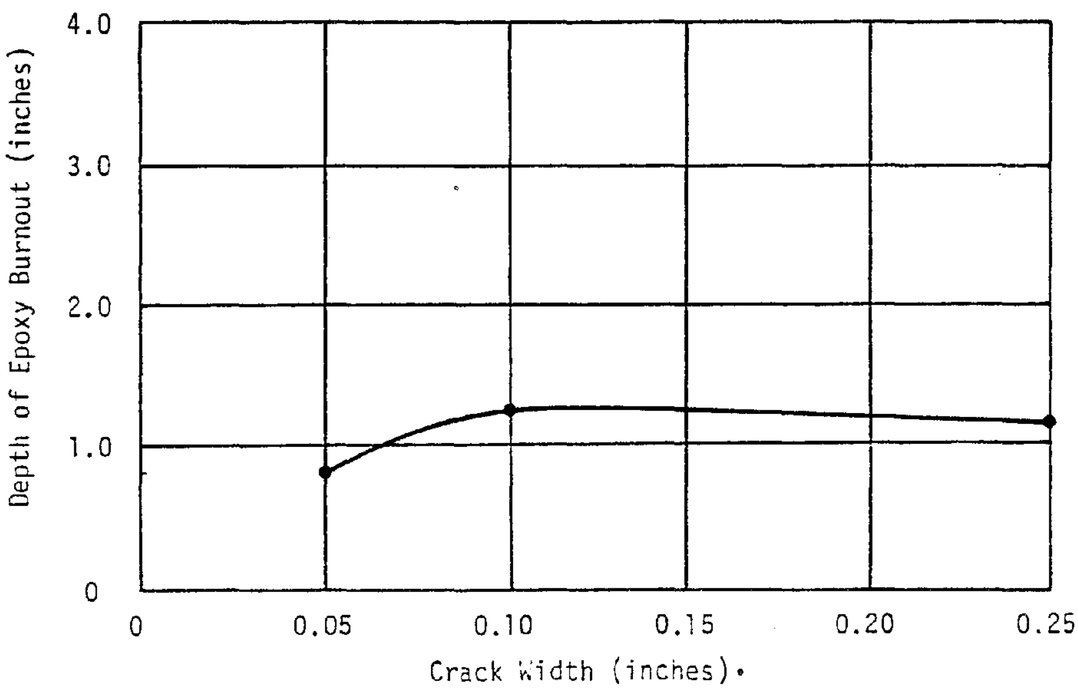

Fig. J-2b: Average Depth of Epoxy Burnout as a Function of Crack Width

REPAIR OF BOND IN R/C STRUCTURES BY EPOXY INJECTION

by

Andrew D. Cowell,[1] Egor P. Popov,[2] and Vitelmo V. Bertero,[2]

Synopsis.--Two experiments were performed to determine the effectiveness of epoxy injection repair in restoring bond of deformed reinforcing bars. Test specimens were designed to simulate the bond deterioration found in the interior beam-column joints of a reinforced concrete ductile moment-resisting frame subjected to severe lateral loads such as those expected during major seismic ground motions. Two different epoxies and methods of injection were evaluated. Although the methods could restore sufficient bond strength to allow the application of sustained working stresses to the reinforcing bar, neither method was able to restore the bar's full capacity.

Introduction.--Following a moderate to severe earthquake, the effective repair of damaged structures becomes an immediate problem. One of the techniques used to restore stiffness and/or strength of earthquake-damaged reinforced concrete structures is epoxy injection [1]. When properly performed, epoxy injection can restore the continuity of cracked concrete. Experiments have shown that epoxy-repaired cracks are stronger than the surrounding concrete, i.e., new cracks will not form in the repaired cracks [2,3]. However, because reinforced concrete is a composite of steel reinforcing and concrete, a mechanical characteristic essential to a reinforced concrete structure is the developing of sufficient bonding or stress transfer between the component materials. Although epoxy injection can restore the continuity of concrete when cracks are within prescribed limits (> 0.1 mm, < 5 mm) [4], tests performed at Berkeley [2,3] and elsewhere [5] have shown that current methods of injection fail to restore bond completely. For reinforced concrete ductile moment-resisting frames (DMRF's), where seismic excitations have caused severe slippage of the beams' main reinforcing bars in beam-column joints, the need for improving the technique of epoxy injection as a means of restoring bond is clear. This paper addresses this need by evaluating the effectiveness of two different methods of epoxy injection in the repair of bond.

Bond Deterioration in Beam-Column Joints.--The problem of loss of bond in beam-column joints was demonstrated in some tests on subassemblages carried out at Berkeley [6]. Typical results for one specimen, before and after repair by epoxy injection, are shown in Fig. 1. The virgin specimen was subjected to a series of pseudo-static load reversals and suffered significant degradation in stiffness after just one cycle of full reversal. The main reason for this behavior was the slippage (pull-out and push-in) of the beams' main longitudinal reinforcing along the column joint. The specimen was then repaired by injecting Concresive 1050-15 epoxy resin [7] using an in-head mixing pump for injection (see reference 4 for details of this method). Upon initial reloading (region

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[1]Research Assistant, University of California, Berkeley.

[2]Professor of Civil Engineering, University of California, Berkeley

OA of Fig. 1), the strength and stiffness of the repaired specimen resembled that of its virgin state. However, as soon as the working load was exceeded (point A), new cracks started to develop, and there was a decrease in the strength and stiffness (region AB) of the specimen in comparison with the original state. It is believed that this decrease in stiffness was due primarily to premature slippage of the beams' main reinforcing bars along the development width of the column. Results of these tests suggest that epoxy injection cannot fully restore bond along the development length in a beam-column joint.

The importance of this sudden failure of development bond when stresses in the main beam reinforcing bars exceed the working stress level can be quantitatively illustrated by an estimation of the fixed-end rotation θ_{FE} that can occur at a column face due only to the pull-out of the beam bars, as in Fig. 2. Considering a beam depth d-d' of 510 mm, a column width of 635 mm, and #8 beam bars of Grade 60 (414 MPa) steel loaded to working stress,

$$\theta_{FE} = [414 \times 0.4/(207 \times 10^3)] \times 635/510 = 0.001 \text{ rad}$$

This fixed-end rotation by itself represents 50% of the story drift index recommended as acceptable for reinforced concrete structures under lateral load (0.002) [8] and 20% of the story drift index recommended under UBC (Uniform Building Code) design seismic forces [9]. Thus, the fixed-end rotations add significantly to the deformations of the frame by decreasing the joint stiffness and thereby softening the overall frame response.

It should be noted that it is possible to minimize or avoid the problems created by slippage of the beams' main bars by specially detailing the reinforcing in the beam so that the critical regions (plastic hinges) are moved away from the column face [10]. However, present seismic design provisions for reinforced concrete DMRF's do not resort to this solution, leaving the development bond in the joints as the weakest element in the entire frame. For all existing moment-resisting frames, as well as those that will, undoubtedly, be designed and constructed without avoiding the above problem, there is a need to consider the possibility of an improved method of epoxy injection that might effectively repair bond. The study reported herein is an attempt to investigate this possibility; it represents part of a comprehensive experimental and analytical study of the mechanisms of bond deterioration and their effect on seismic design.

Test Specimens.--Rather than working with an entire beam-column subassemblage, it was decided to simplify the test specimen as a simulated column section that consists of a reinforced concrete block cast around a single transverse beam bar. A simplification was necessary at this stage of the study to remove the transverse shear present in a complete beam-column joint. Also, for reasons of simplicity, a single bar was cast in the simulated column rather than the usual row of top or bottom beam bars.

The simulated column is 250 mm thick and 1,150 mm high; the column width is 635 mm in which a #8 (25 mm) bar was cast to simulate the beams' longitudinal reinforcement. The column section contains eight #7 (22 mm) bars for a ρ of 1.92%, which is close to the minimum 2% required by code [9]. Overlapping pairs of #4 (13 mm) ties at 100 mm on center provide good confinement for the concrete. Normal weight aggregate concrete was used with a cylinder strength of approximately 31 MPa during the test of the original specimens; at retesting, after repair, concrete strengths were 38 MPa for Specimen 7 and 35 MPa for Specimen 9. All reinforcing bars were of grade 60 (414 MPa) steel and ribbed deformation pattern. The average yield strength of the #8 transverse bars was 480 MPa with an ultimate strength of 752 MPa.

Test Setup and Instrumentation.--A plan view of the overall experimental arrangement is shown in Fig. 3. Specimens are held in place by the frictional resistance of heavy metal tie-down straps that are bolted to horizontal supports. These straps are lightly prestressed to the specimen to avoid stress concentrations that might affect the bond characteristics of the concrete surrounding the bar. Hydraulic rams are capable of applying equal push-pull forces of 530 kN to the ends of the bar. Each specimen is instrumented to monitor movement (pull-out and push-in) of the bars at the ends of its embedment length as well as strains along the length of the bar. Pull-out and push-in are measured with reference to the column centerline. Strains are measured by affixing post-yield strain gages into two diametrically opposite narrow grooves machined along the length of the bar.

Experiments on Virgin Specimens.--The two specimens considered in this paper were subjected to different loading programs in their original tests.

Specimen 7.--This specimen underwent a nominally monotonic test. Equal tensile and compressive forces were applied to opposite ends of the bar. The loading program consisted of a series of small cycles up to a working stress level of 165 MPa followed by a monotonic excursion whose purpose was to yield the bar simultaneously on both tensile and compressive ends, eventually pulling the bar through the column. The load was then reversed in order to bring the specimen back to the zero displacement position.

Specimen 9.--The original loading program consisted of cycles of full load reversals of increasing intensity up to failure. Forces were simultaneously applied in the push-pull manner. Small cycles, as in the monotonic test, were followed by a series of three cycles, the first of which caused yielding of the bar. Groups of three cycles of increasing severity followed until only a frictional resistance of 20 MPa remained for the last cycle of the original test (Fig. 7).

Repair of Specimens.--It is known from the epoxy repair of cracks that any debris within the fissure can negate effective adhesion between crack faces [11]. This is partly a consequence of the epoxy's viscosity, which prevents it from penetrating cracks smaller than about 0.1 mm. Either a more penetrating epoxy resin or a more efficient method of injection is required for a repair better than that in the previously described beam-column subassemblage.

Specimen 7.--A first attempt at improved repair was to use a lower viscosity epoxy on Specimen 7. Adhesive Engineering Concresive 1380 epoxy [4] had a lower viscosity and, presumably, greater penetrability than the Concresive 1050-15 used in the earlier repair of the beam-column subassemblages [3]. Standard injection procedure for this epoxy, using an in-head mixing pump, was followed [4]. This particular technique seems to perform well in repairing cracks that have developed along a flat surface where a good seal can be made between the injection nozzle and the crack opening. The uneven surface of the spalled area around the end of the bar caused a poor seal during injection, making effective epoxy penetration unlikely. This was confirmed when a specimen subjected to the same loading program was epoxy injected by the same procedure and then sectioned with a diamond saw. The epoxy penetration at the ends of the bar's embedment length was no more than 75 mm. Moreover, epoxy could not be injected through surface cracks to reach the damaged area around the bar since good column confinement prevented radial splitting cracks from extending from the bar to the surface of the specimen.

Specimen 9.--A different technique, as well as a lower viscosity epoxy, Sikastix 350 [12], was used in the repair of this specimen. The method involves batch mixing followed by injection from a pressurized vessel [4]. Small plastic fittings are epoxied to the openings in the surface sealer to achieve an improved fit between the injection nozzle and the crack. For this specimen, injection was prolonged at each injection site to allow the maximum possible penetration of the epoxy.

Both specimens were allowed to cure well in excess of their full curing times of 2-3 days at 25°C [7,12].

Experiments on Repaired Specimens.--Both repaired specimens were tested under a monotonic loading program of simultaneously applied push-pull forces at either end of the test bar. Experimental results are described below.

Specimen 7.--Figure 4 shows the relationship of stress in the test bar to pull-out at the column face for the specimen in its original state as well as after epoxy repair. The maximum stress sustained by the repaired specimen test bar was 269 MPa, compared to 655 MPa in the original test. The epoxy repair, though exceeding working stress level (166 MPa), did not allow the bar to reach yield strength (480 MPa). The initial stiffness was slightly less in the repaired specimen than in the virgin specimen.

Distributions of strain along the test bar at increasing load levels are given in Fig. 5 for the original specimen and Fig. 6 for the repaired specimen. The abruptly decreasing strains at the left of Fig. 6 indicate that a major part of the tensile stress in the bar is transferred to the concrete in this region, whereas in the original test the strains are more uniformly distributed over the embedment length.

It appears that the epoxy injection penetrated no more than 50 mm along the remaining embedment length (note that the first 75 mm of the embedment length surrounding the end of the bar had spalled off during the original test).

Specimen 9.--Figure 7 shows the monotonic loading of the epoxied specimen along with envelope curves of the original cyclic test. The maximum stress level in the test bar after repair was about 310 MPa, not sufficient to cause yielding. The initial stiffness after repair was slightly less than in the virgin specimen; moreover, the epoxy-repaired specimen showed a rapid decrease in stiffness until the bar pulled through.

Strain distributions for the test bars of the original and repaired specimens, Figs. 8 and 9, show a gradual decrease of strain along the embedment length. This indicates a deeper penetration of the epoxy along the embedment length than was experienced in Specimen 7. However, only a small increase (15%) in maximum stress was obtained.

Implications of Test Results in Seismic-Resistant Design.--If a structure repaired by the methods described for the test specimens experiences another earthquake strong enough to induce stresses of about 300 MPa, it will undergo a loss of bond resistance and a considerable decrease in lateral stiffness as a consequence of the fixed-end rotation that will occur. The magnitude of θ_{FE} under a practically constant M (considerably lower than M_y) further depends on the state of the concrete in the beam located on the side of the column where the beam bars are in compression. The effect of this sudden drop in lateral stiffness on the overall structure depends on the ground motion characteristics of the earthquake and the dynamic characteristics of the structure. It should also be recognized that poor technique, adverse curing conditions, and improper epoxy formulations could easily reduce the effectiveness of the bond repair. Furthermore, it is difficult to judge the extent of repair, especially within a column joint; inspection by core drilling in a reinforced concrete member is not practical. Therefore, the variability and uncertainty of epoxy injection must be considered.

Conclusions and Recommendations.--At this initial stage of the experimental study of bond repair of joints, the following conclusions and recommendations may be offered:

1. The epoxy materials and techniques tested could not repair the bond sufficiently to allow the bar to develop its yield capacity. This deficiency may make the repaired joint ineffective under subsequent seismic loadings. Other means of strengthening and stiffening, in addition to epoxy repair, must be considered if a high level of strength and/or stiffness is required. Suggestions for minimizing and avoiding the problem of bond deterioration have been given [6,10].

2. The problem with epoxy repair of severely deteriorated bond is twofold: the material must be able to penetrate along the damaged embedment length as well as to permeate the finely ground concrete between the bar deformations. Test results suggest that greater epoxy penetration along the damaged length does not significantly increase restored bond strength. The inability of the epoxy to reconstitute the powdered concrete around the bar may account for the low bond capacity obtained. New formulations of epoxy adhesives should address this specific problem.

Acknowledgements.--The reported work was supported by NSF under Grant No. ENV-76-04263, for which the authors are most grateful.

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4. Warner, J., "Methods for Repairing and Retrofitting (Strengthening) Existing Structures," preprint from the Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, University of California, Berkeley, July 1977.

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6. Bertero, V. V. and E. P. Popov, "Seismic Behavior of Ductile Moment-Resisting Reinforcing Concrete Frames," Special Publication SP-53, ACI, 1977, pp. 247-291.

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9. Uniform Building Code, International Conference of Building Officials, Pasadena, California, 1976 ed.

10. Popov, E. P., et. al., "On Seismic Design of R/C Interior Joints of Frames," preprint from the Sixth World Conference on Earthquake Engineering, New Delhi, India, January 1977, Vol. 5, 191-195.

11. Hanson, R. D., "Repair and Strengthening of Reinforced Concrete Members and Buildings," preprint form the Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, University of California, Berkeley, July 1977.

12. Technical Bulletin HMLV-0677, Sika Chemical Co., Lyndhurst, New Jersey.

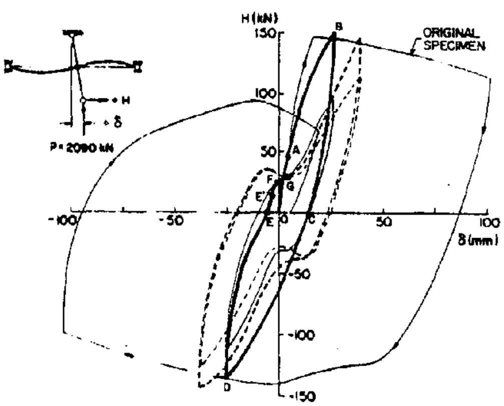


Fig. 1 Hysteretic Behavior of Repaired Subassemblage

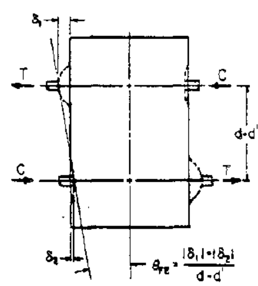


Fig. 2 Fixed-End Rotations at an Interior Column

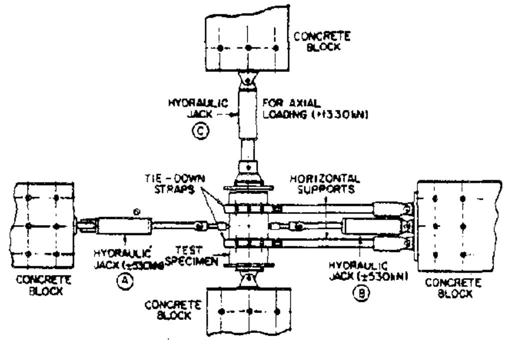


Fig. 3 Experimental Arrangement

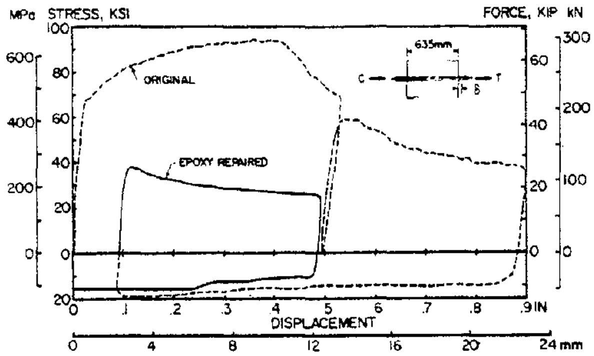


Fig. 4 Hysteretic Behavior of Repaired Specimen 7

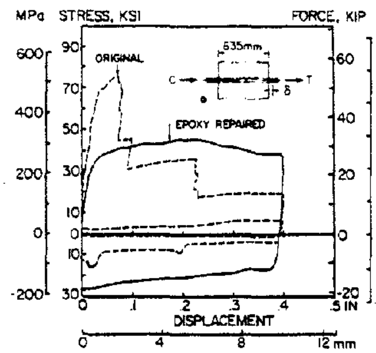


Fig. 7 Hysteretic Behavior of Repaired Specimen 9

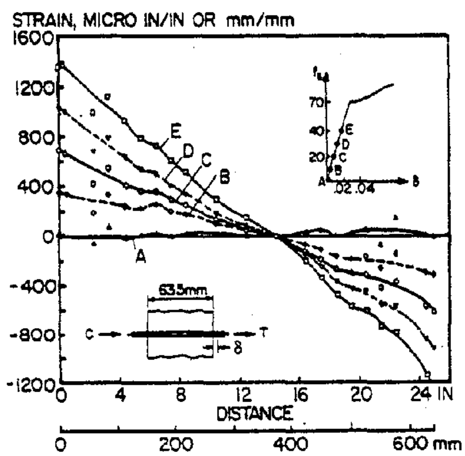


Fig. 5 Strain Distributions Along Bar of Specimen 7, Original Test

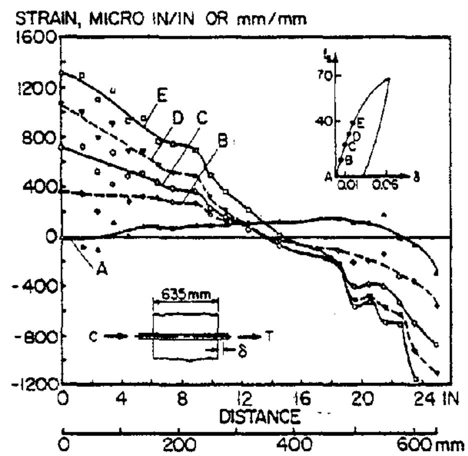


Fig. 8 Strain Distributions Along Bar of Specimen 9, Original Test

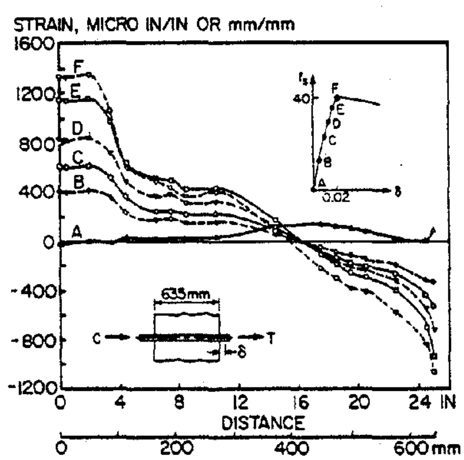


Fig. 6 Strain Distributions Along Bar of Repaired Specimen 7

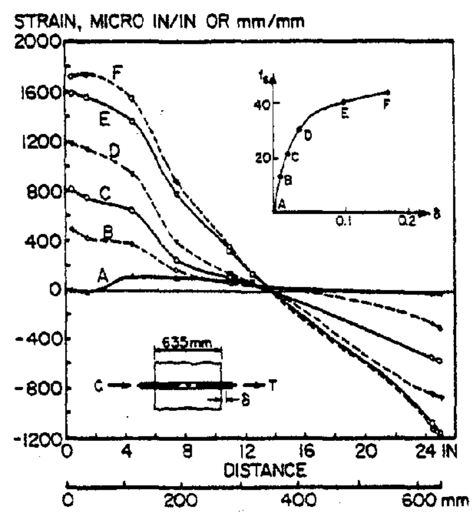


Fig. 9 Strain Distributions Along Bar of Repaired Specimen 9

RESEARCH PROJECT ON REPAIR AND RETROFIT OF BUILDINGS

1. Special Applied Research Project

   1977 - 1979 .... Reference A.

2. Cooperative Research and Development Project

   1980 - 1984 .... Reference B.

3. U.S.-JAPAN Cooperative Research Program Utilizing Large-Scale Testing Facilities

   1979 - 1981 .... Reference C.

Arranged by:

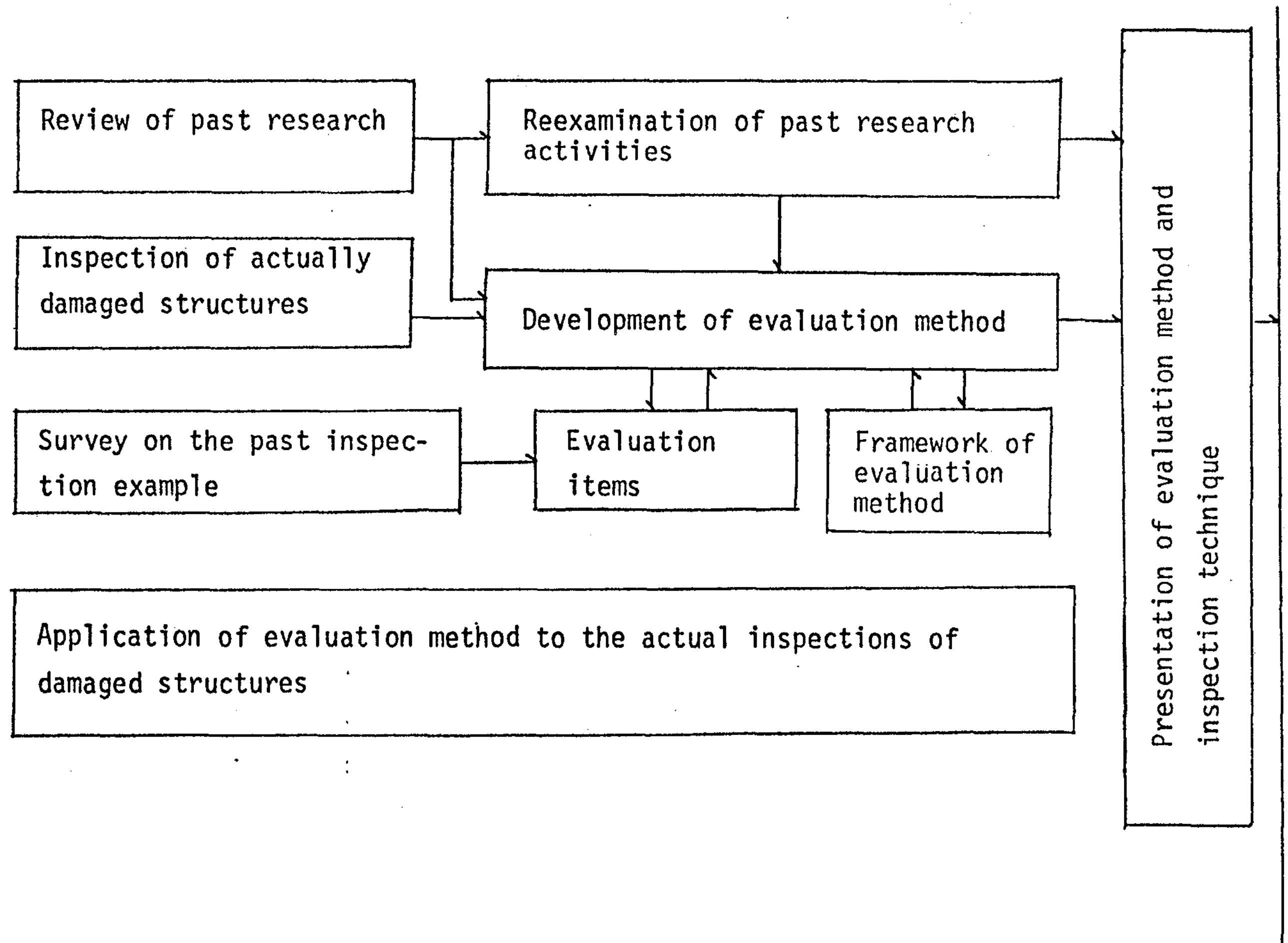
K. Nakano, Dr. Eng.
S. Okamoto
T. Kubota
T. Kaminosono

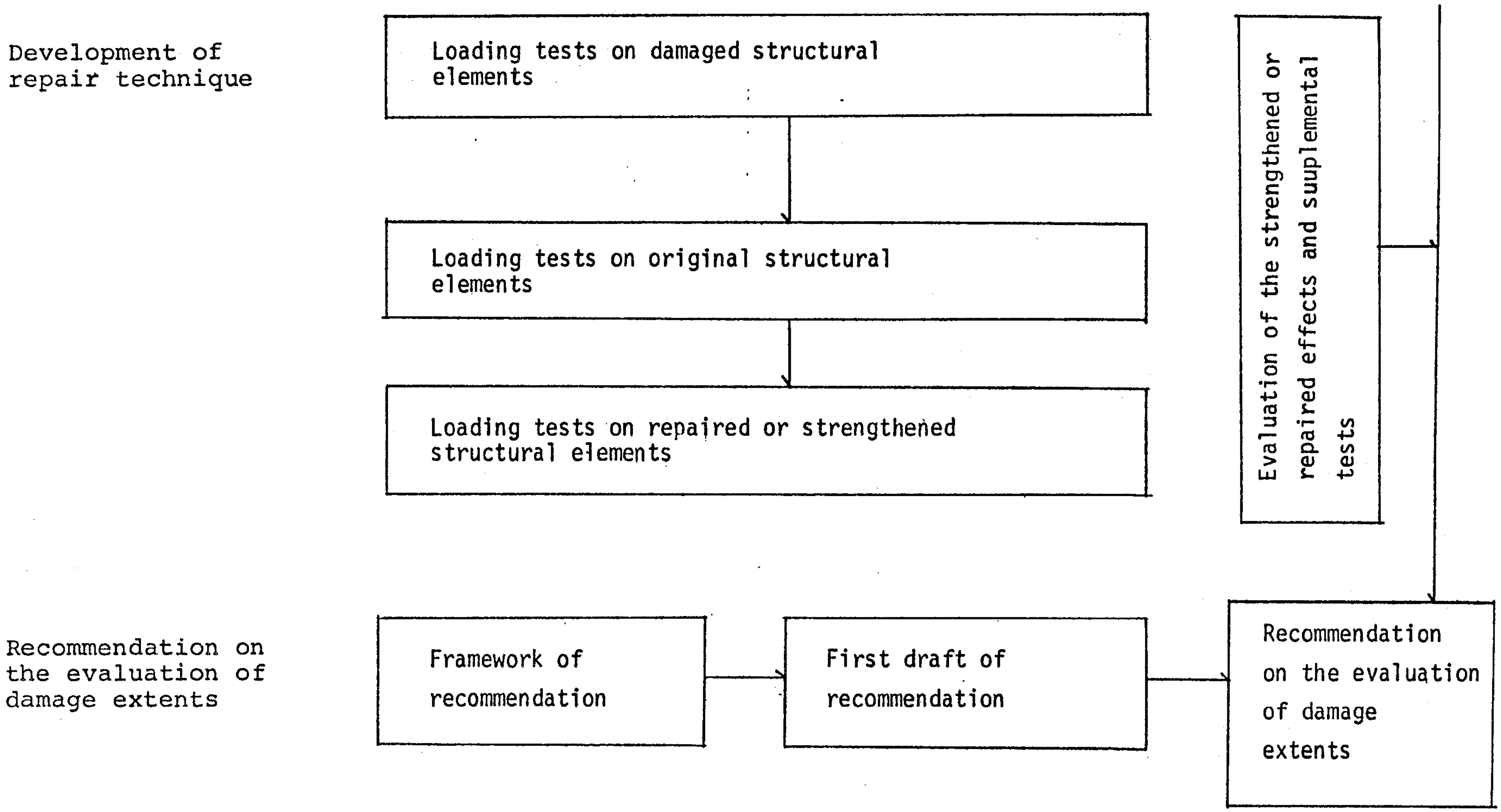
BUILDING RESEARCH INSTITUTE
MINISTRY OF CONSTRUCTION
JAPANESE GOVERNMENT

Reference A. Special Applied Research Project "Retrofit technique of structural performance of damaged structure due to earthquake, fire, gas explosion, etc."

Research Items:

Evaluation method for damage extent





Development of repair technique
Loading tests on damaged structural elements
Loading tests on original structural elements
Loading tests on repaired or strengthened structural elements
Evaluation of the strengthened or repaired effects and suuplemental tests
Recommendation on the evaluation of damage extents
Framework of recommendation
First draft of recommendation
Recommendation on the evaluation of damage extents

Example 1 : Loading Tests on Repaired Reinforced Concrete
Beams after Fire Endurance Test

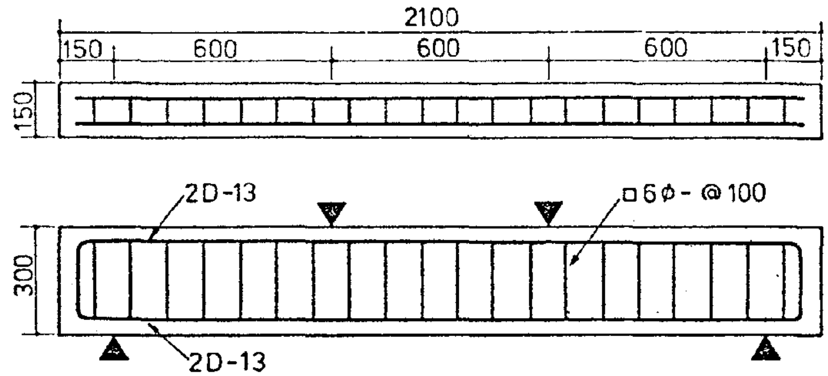
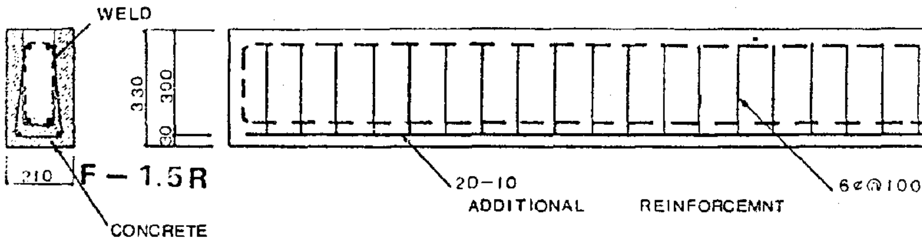
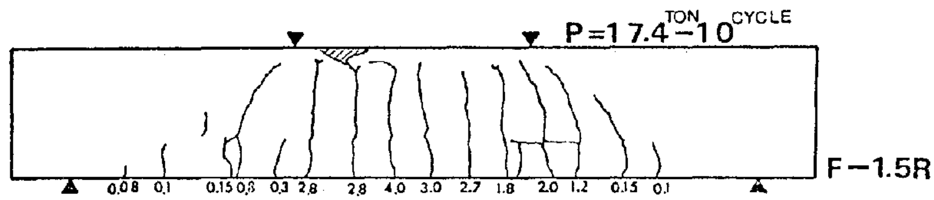


FIG.1 TEST SPECIMEN

Table 1 Test Specimen and Method of Repaire

Test specimen	Heating history before repair	Method of repair
F 1.5	1.5 hours fire resistance test	Strengthning by additional tension reinforcement and re-cast of cover concrete
F 1.0	1.0 hours fire resistance test	Strengthening by expanded metal and re-cast of cover concrete


WELD
330
300
30
210
F − 1.5R
CONCRETE
2D−10
ADDITIONAL REINFORCEMNT
6¢@100


P=17.4 TON −10 CYCLE
F−1.5R
0.08 0.1 0.15 0.3 0.3 2.8 2.8 4.0 3.0 2.7 1.8 2.0 1.2 0.15 0.1

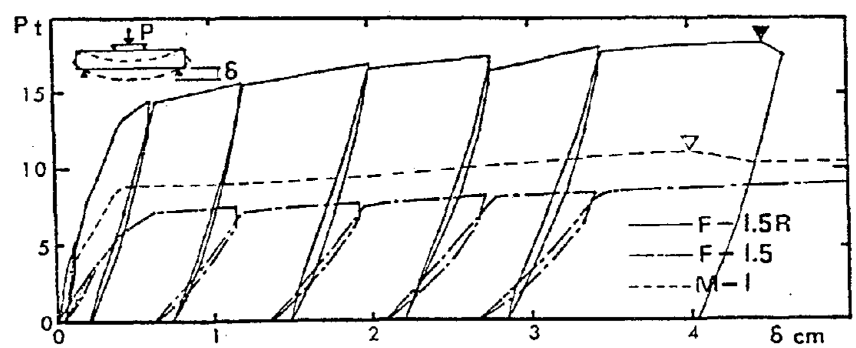
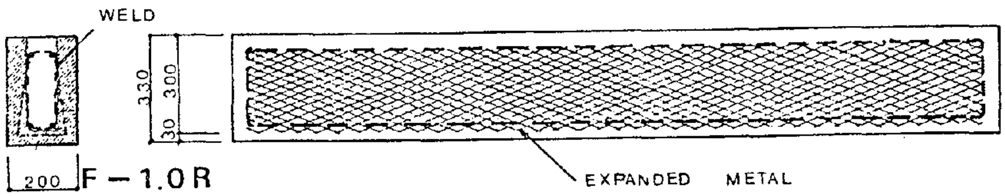
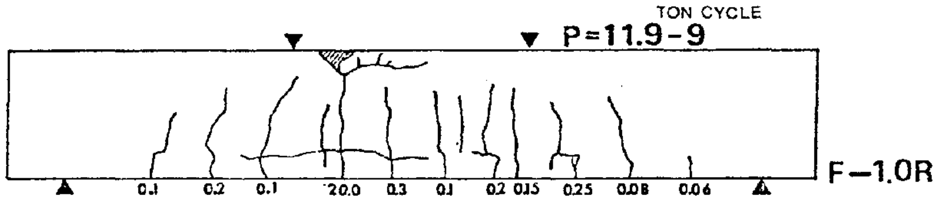

Pt
P
δ
15
10
5
0
0
1
2
3
4
δ cm
F − 1.5R
F − 1.5
M − 1

FIG.2 F-1,5R


WELD
330
300
30
200
F — 1.0 R
EXPANDED METAL


TON CYCLE
P=11.9-9
F—1.0R
0.1
0.2
0.1
20.0
0.3
0.1
0.2
0.15
0.25
0.08
0.06

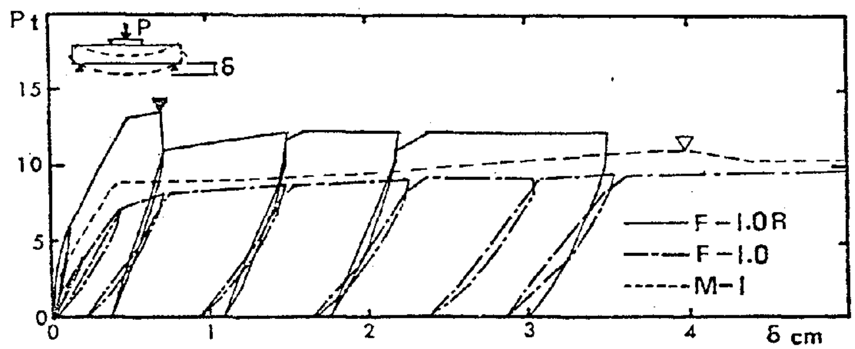

Pt
P
δ
15
10
5
0
1
2
3
4
δ cm
F-1.0R
F-1.0
M-1

FIG.3 F-1.0R

Example 2 : Loading Tests on Repaired Reinforced Concrete Beams after Cracking

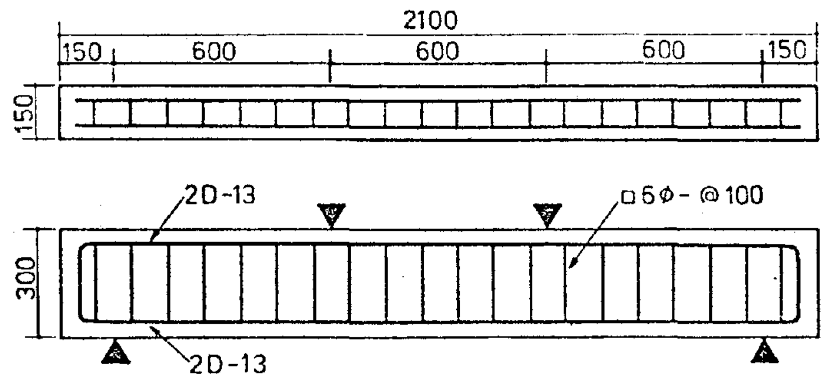


Fig.1 Test Specimen

Table 1. Test Specimen and Methods of Repair

Test Specimen	Deflection history before repair or Strengthening	Method of repair
M-1	Ultimate failure	
M-2	δ ≒50mm	Strengthening by additional longitudinal tension reinforcement and re-cast of cover concrete
M-3	$2\delta y$ (10mm)	Repair by epoxy for the cracks more than 1.0mm width
M-4	$2\delta y$	Strengthening by additional longitudinal tension reinforcement
M-5	$3\delta y$ (15mm)	Same as M-4
M-6	$3\delta y$	Same as M-2

δ : Mid span deflection

δy: Yield deflection

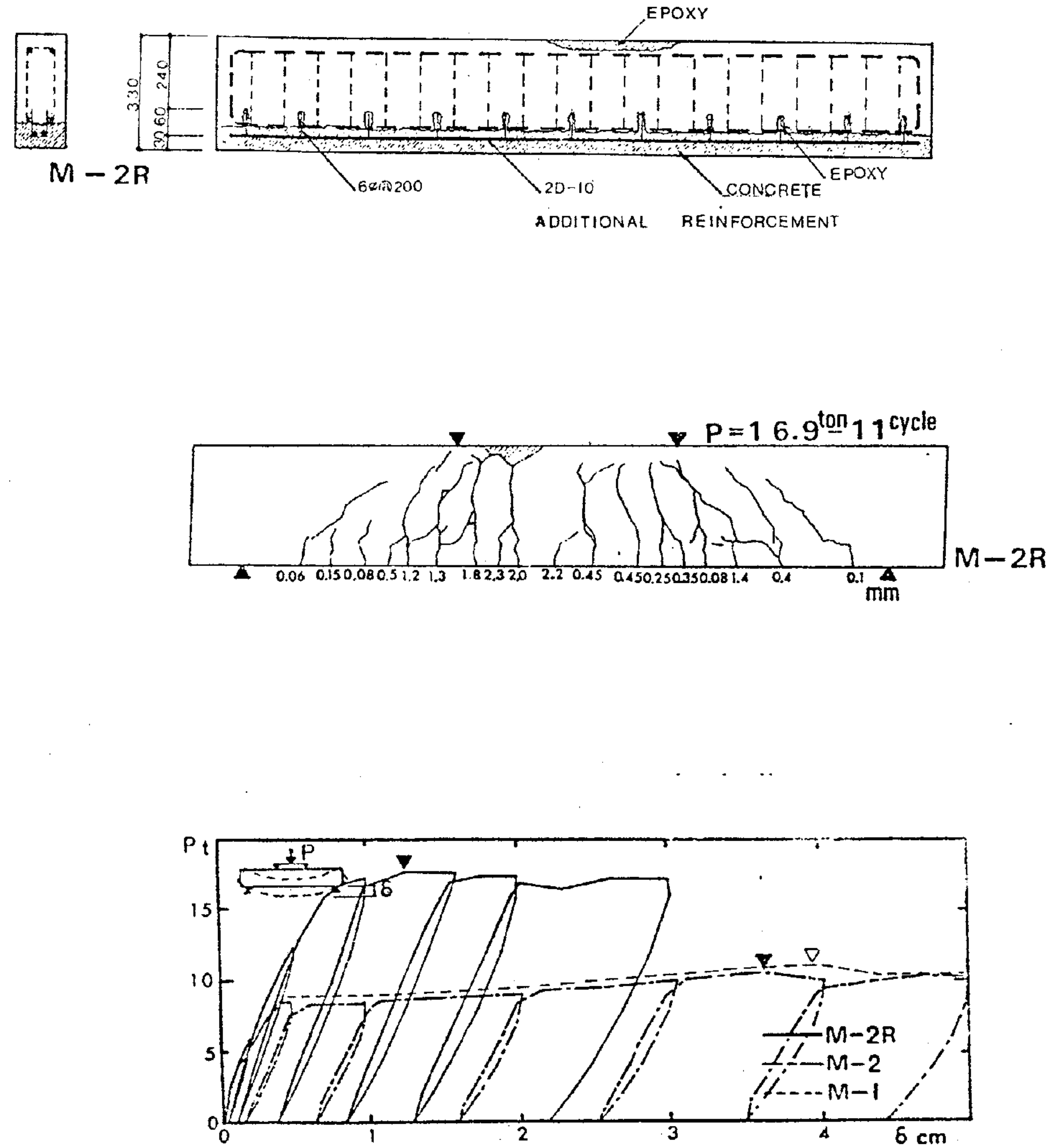

EPOXY
M − 2R
330
240
60
30
6φ@200
2D−10
CONCRETE
EPOXY
ADDITIONAL REINFORCEMENT
P=16.9ton 11cycle
0.06 0.15 0.08 0.5 1.2 1.3 1.8 2.3 2.0 2.2 0.45 0.45 0.25 0.35 0.08 1.4 0.4 0.1
mm
M−2R
Pt
P
δ
15
10
5
0
0
1
2
3
4
δ cm
M−2R
M−2
M−1

FIG. 2 M− 2R

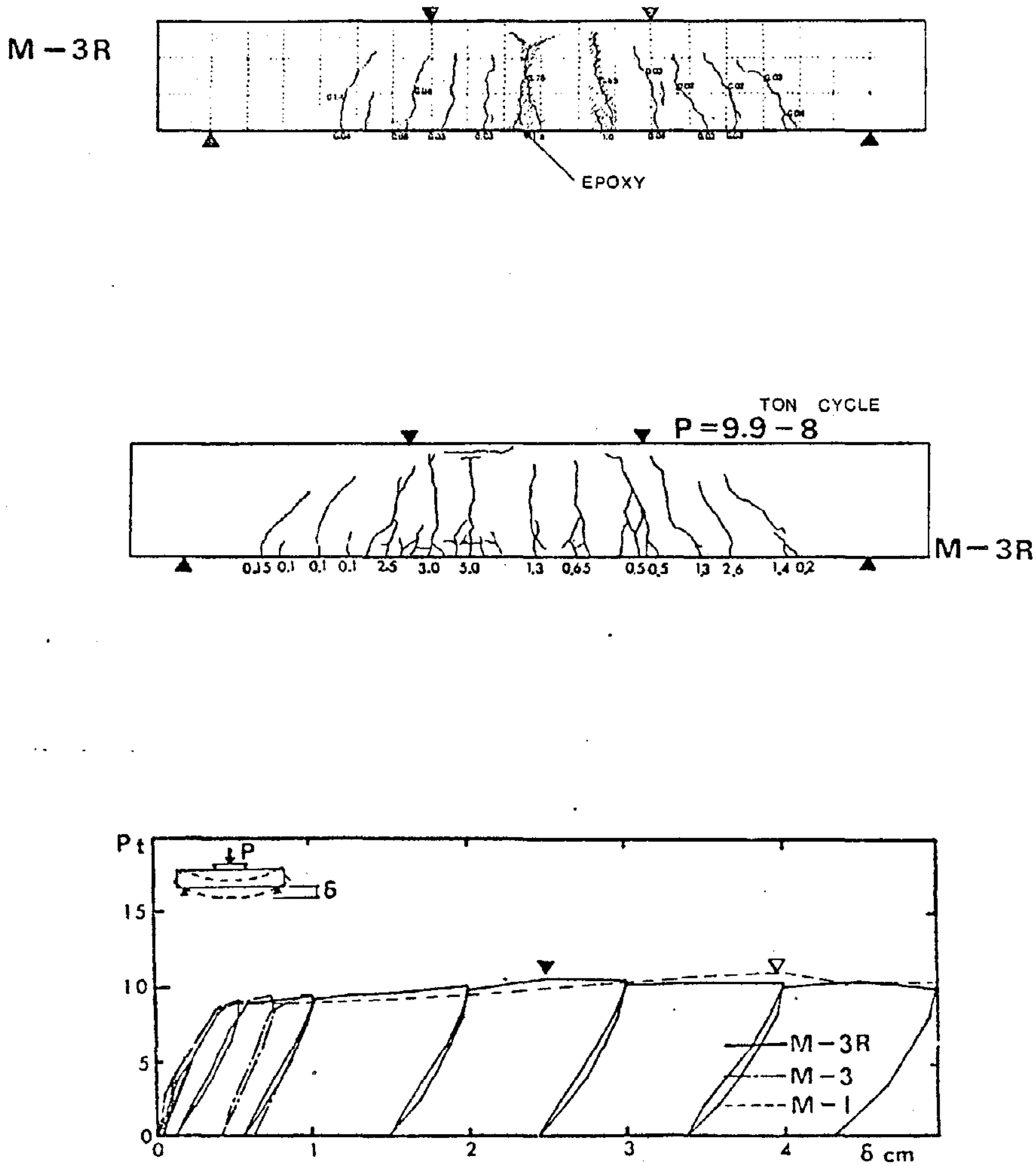

M-3R
EPOXY
TON CYCLE
P=9.9-8
M-3R
0.15 0.1 0.1 0.1 2.5 3.0 5.0 1.3 0.65 0.5 0.5 1.3 2.6 1.4 0.2
Pt
P
δ
15
10
5
0
0
1
2
3
4
δ cm
M-3R
M-3
M-1

FIG. 3 M-3R

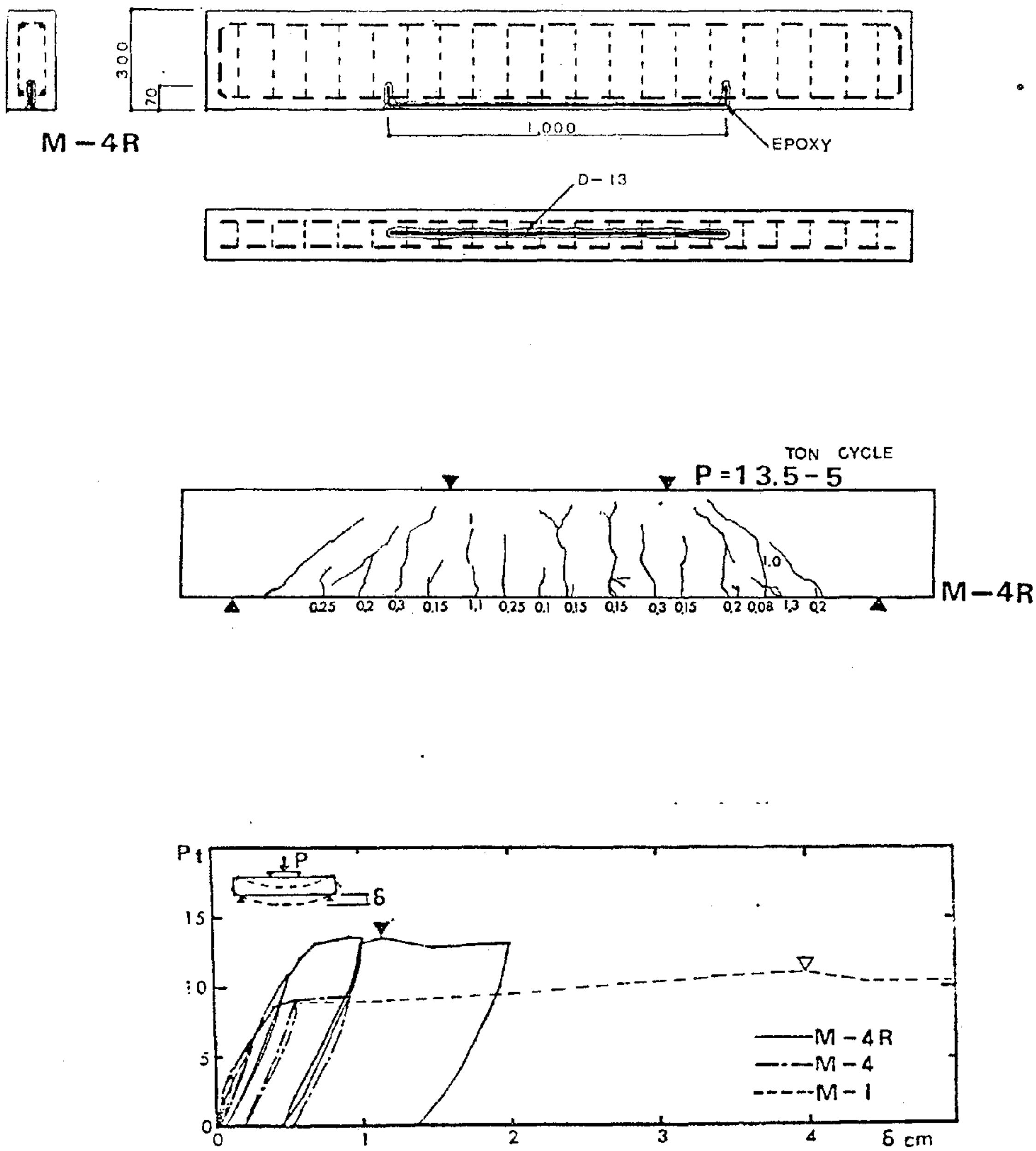
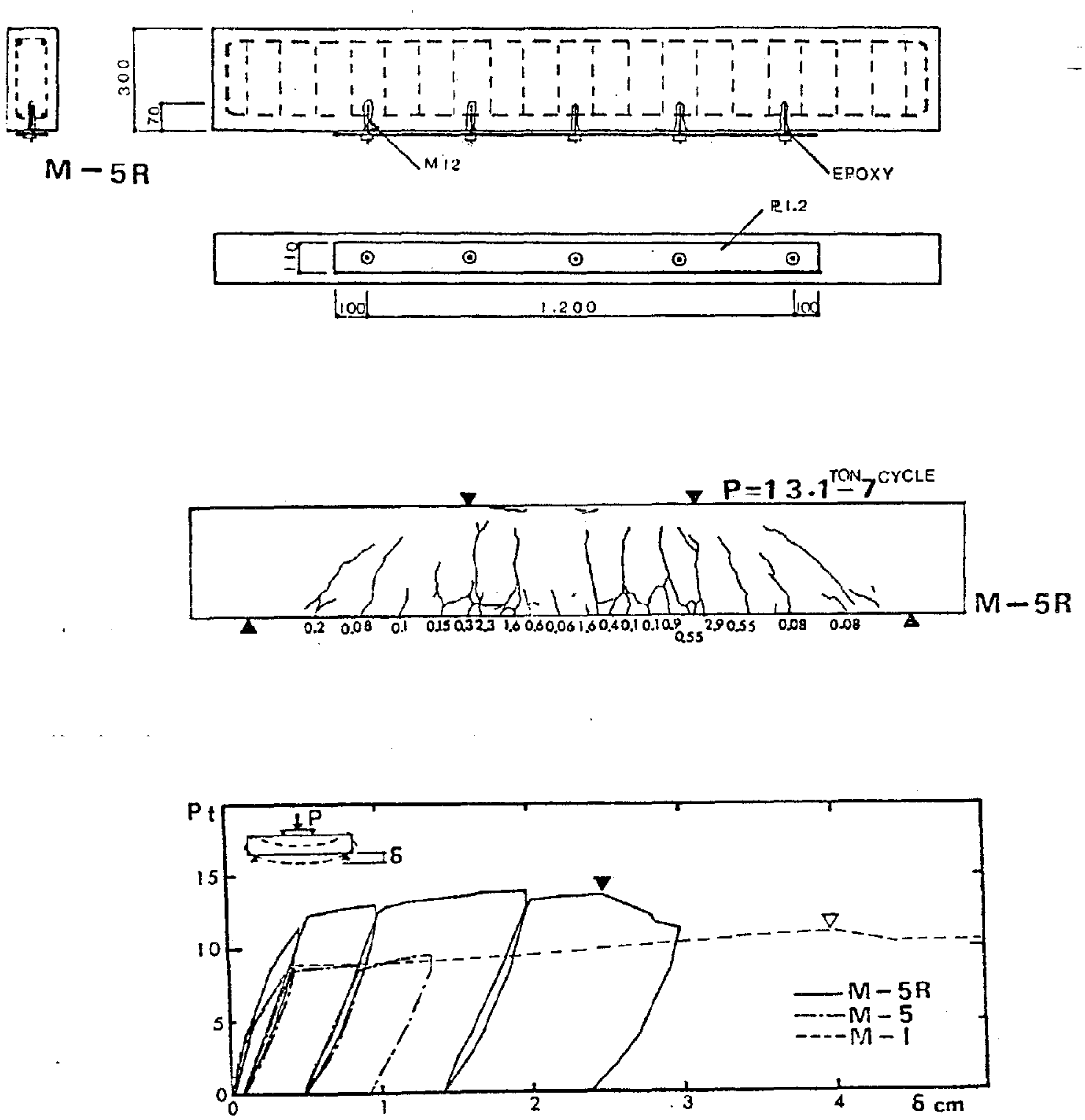

M−4R
300
70
1,000
EPOXY
D− 13
TON CYCLE
P =13.5−5
0.25 0.2 0.3 0.15 1.1 0.25 0.1 0.15 0.15 0.3 0.15 0.2 0.08 1.3 0.2
1.0
M−4R
Pt
15
10
5
0
0 1 2 3 4 δ cm
P
δ
M −4R
M −4
M − I

FIG. 4 M-4R



300
70
M-5R
M12
EPOXY
PL1.2
110
100
1.200
100
P=13.1 TON 7 CYCLE
M-5R
0.2
0.08
0.1
0.15
0.32
2.3
1.6
0.6
0.06
1.6
0.4
0.1
0.1
0.9
0.55
2.9
0.55
0.08
0.08
Pt
P
δ
15
10
5
0
0
1
2
3
4
δ cm
M-5R
M-5
M-1

FIG.5 M-5R

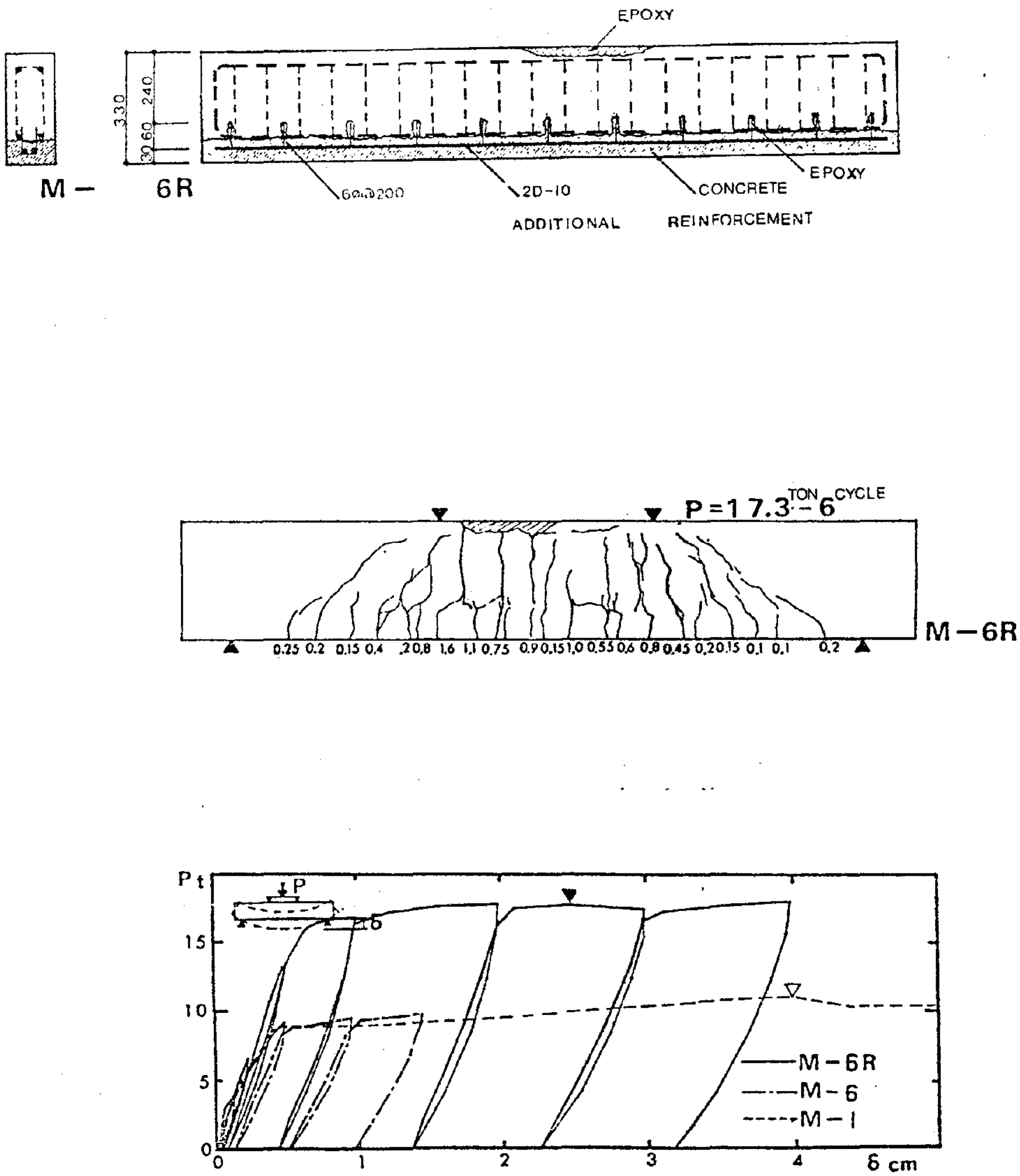

EPOXY
330
240
60
30
M – 6R
6φ@200
2D-10
CONCRETE
EPOXY
ADDITIONAL REINFORCEMENT
P=17.3 TON - 6 CYCLE
M–6R
Pt
15
10
5
0
0
1
2
3
4
δ cm
P
δ
M-6R
M-6
M-1

FIG.6 M-6R

Reference B. Development of the Improvement Technique for the Durability of Buildings

I. Preservation Technique of Existing Buildings

1. Examination technique for the deterioration of buildings

   (1) Inspection and research on deterioration phenomenon

   (2) Evaluation of the environmental condition for the deterioration of buildings

   (3) Evaluation criteria for the extent of deterioration

2. Development of repair and exchange technique

   (1) Repair and exchange technique of structural elements

   (2) Repair and exchange technique of non-structural elements

   (3) Repair and exchange technique of materials and elements of building equipment

II. Improvement Technique of the Durability of Newly Establishing Buildings

1. Requirements for durability

   (1) Requirements for the durability of building materials and elements

   (2) Research on the design loadings for durability

   (3) Evaluation criteria for the durability of building materials and elements

2. Development of building materials and elements and construction technique to improve durability in buildings

   (1) Development of structural materials and elements and their construction technique

(2) Development of non-structural materials and elements and their construction technique

(3) Development of materials and elements of building equipments

3. Control technique of construction works

III. Evaluation Method on the Preservation Technique and the Improvement Technique of Durability in Buildings

1. Evaluation of the economy for rehabilitation of buildings

2. Evaluation of the economy for improvement of durability in buildings

3. Research on the efficient method of maintenance and management of buildings

IV. Development of Synthetic Technique for Improvement of Durability in Buildings

Reference C.

1. Pseudo dynamic tests on the full scale 2-storied reinforced concrete frame

(1.1) Test specimen Fig. 1 and 2.

(1.2) Method of Strengthening Fig. 3 and 4.

(1.3) Loading Sequence

Pseudo dynamic loading Fig. 5.

|

1978 OFF MIYAGI PREFECTURE EARTHQUAKE
Max. Input Acceleration ; 387 gal

↓

Strengthening

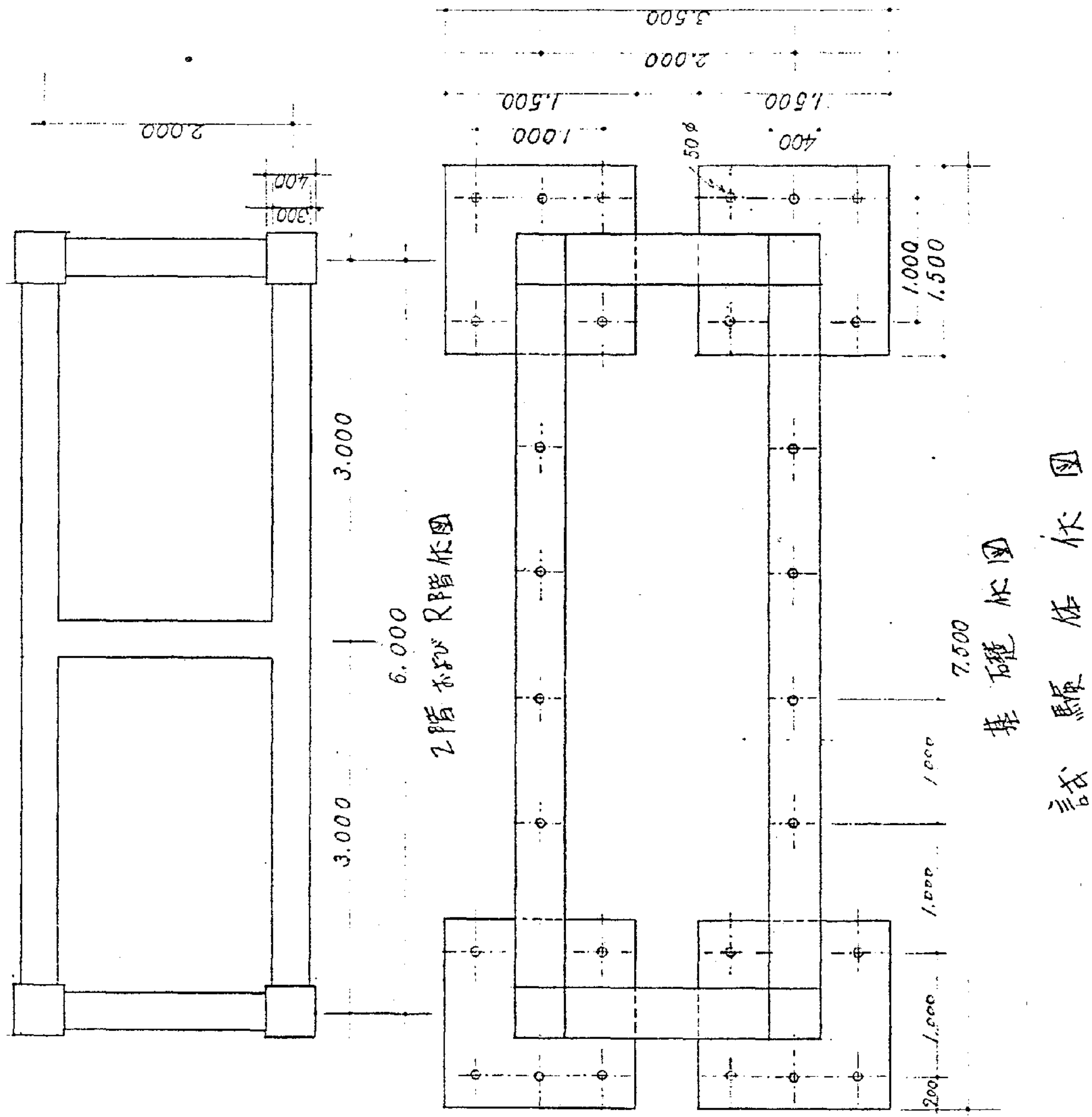
↓

Pseudo dynamic loading

2. Pseudo dynamic tests on the full scale seven storied reinforced concrete frame structure with shear wall

(2-1) Test specimen Fig. 6 and 7.

(2-2) Loading sequence Table 1.



試験体伏図
基礎伏図
2階およびR階伏図
3.500
2.000
1.500
1.000
400
300
50φ
3.000
6.000
7.500
200

Fig. 1

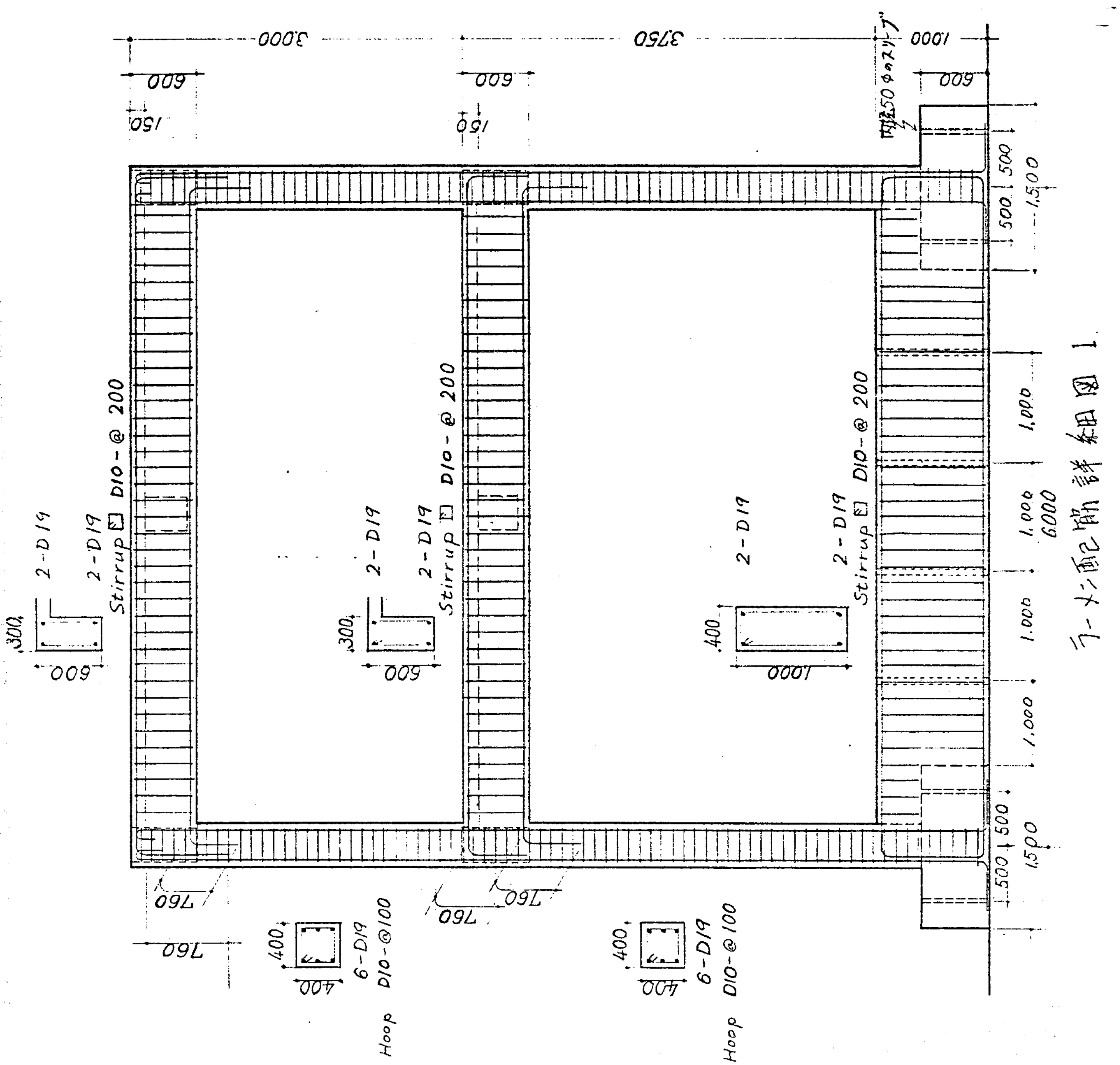

ラーメン配筋詳細図 1
3000
3750
1000
600
150
D10-@200
2-D19
Stirrup
300
400
1000
6,000
1,000
500
1,500
760
6-D19
D10-@100
Hoop

Fig. 2

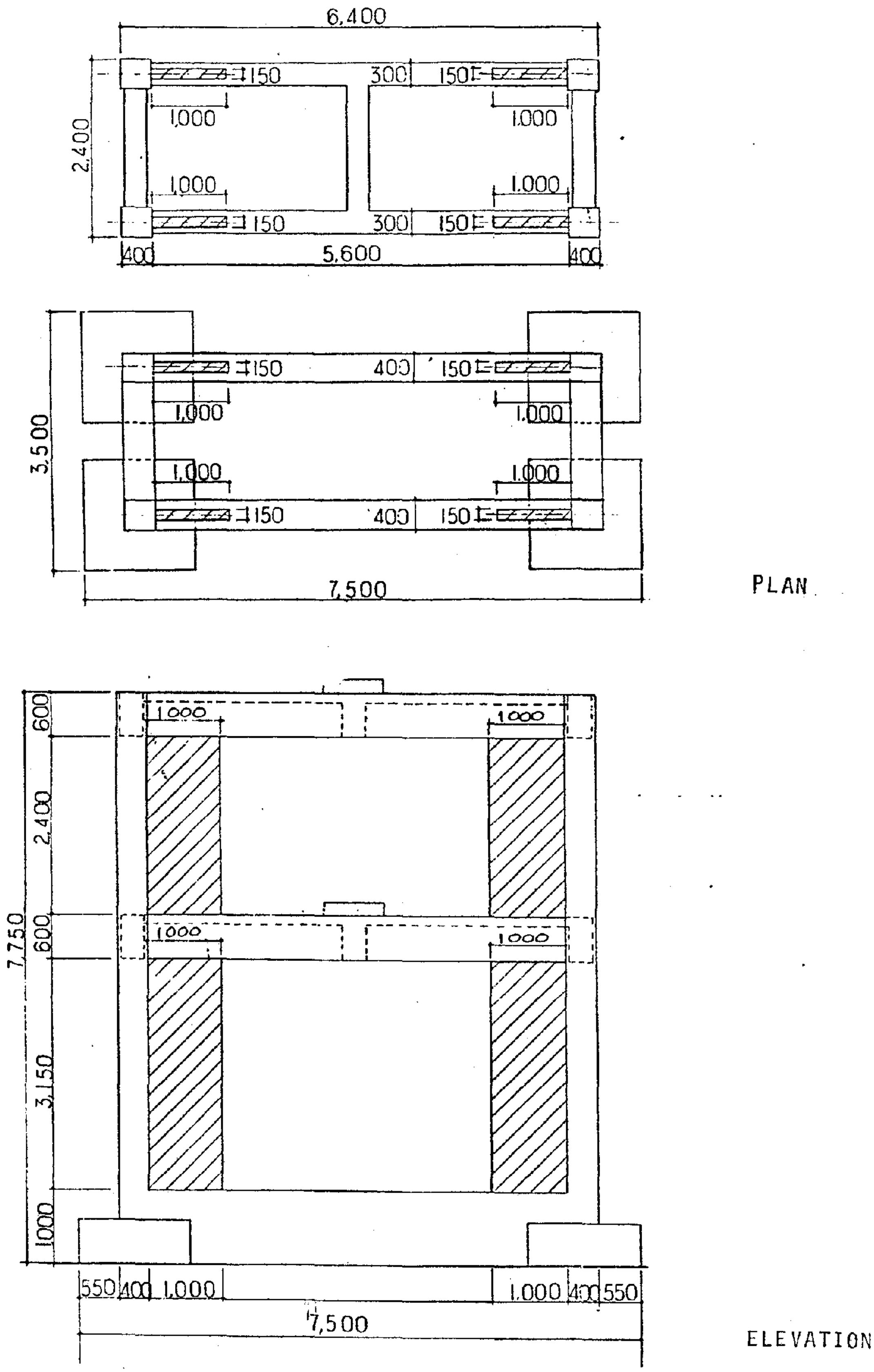


FIG.3 STRENGTHENING BY WING-WALL

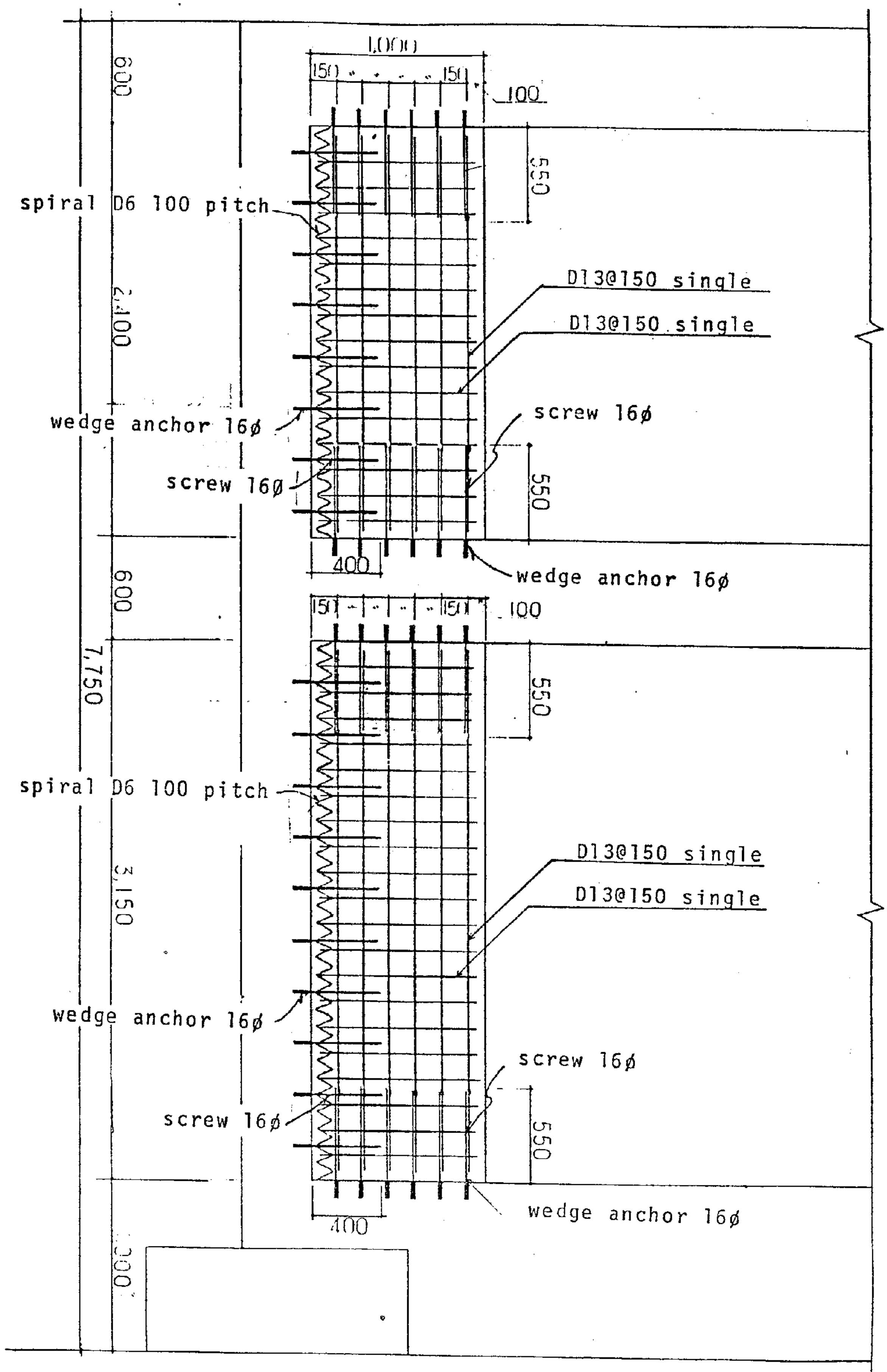


FIG.4 DETAILS OF STRENGTHENING

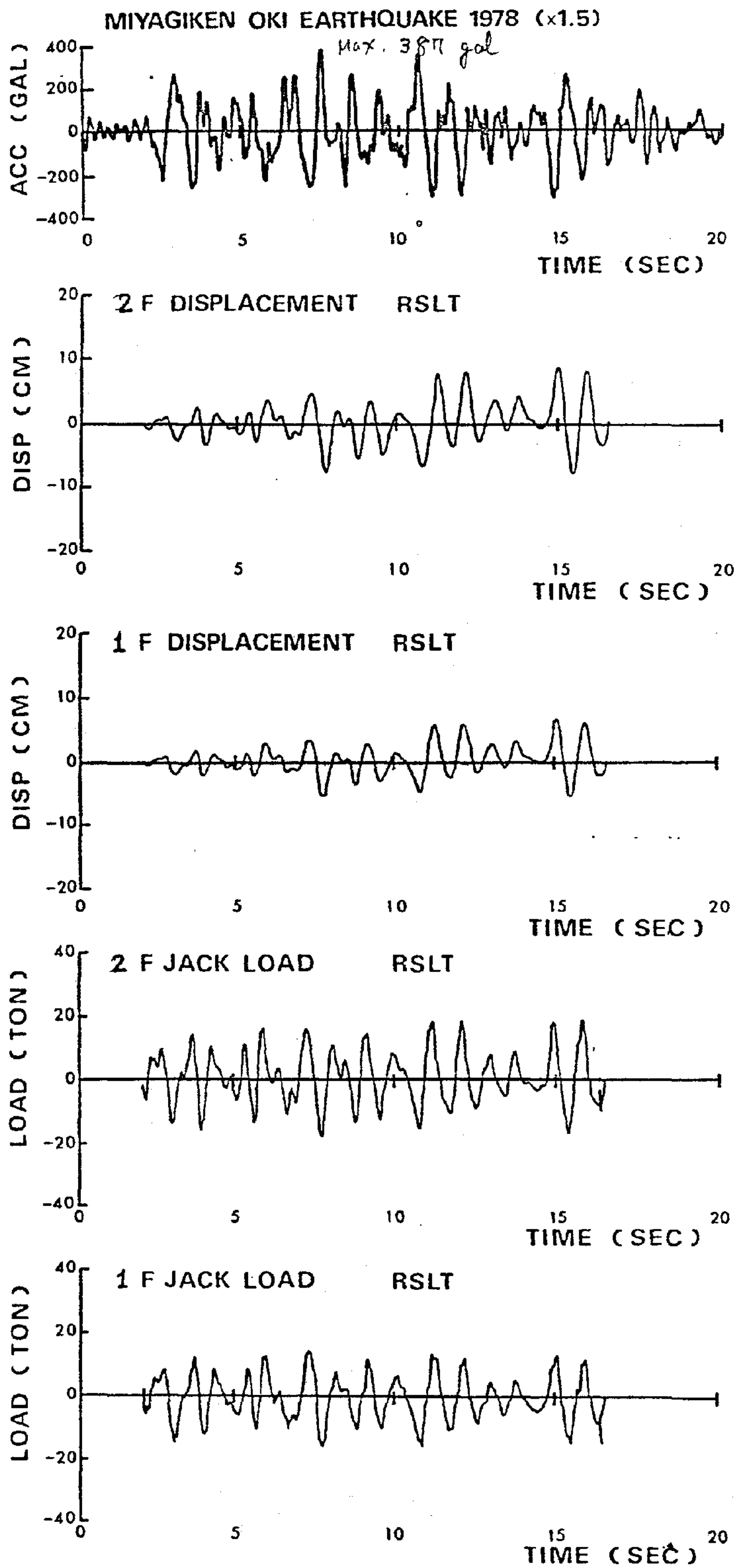


Fig.5 Results of pseudo-dynamic test on 2-storied reinforced concrete frame.

Table 1. Loading Sequence.

Testing	Method Structure	Max. Acceleration of Input Earthquake
F.V.T.	Bare Frame	
P.D.T.		200 gal
F.V.T.		
	Fitting of Non-Structural Member	
F.V.T.		
P.D.T.		200 gal
F.V.T.		
P.D.T.		400 gal
F.V.T.		
	Repair by epoxy	
F.V.T.		
	Strengthening by cast in-place exterior spandrel wall	
F.V.T.		
P.D.T.		400 gal
F.V.T.		
Cyclic Static Loading		
F.V.T.		

F.V.T. Forced Vibration Test

P.D.T. Pseudo Dynamic Test