RETROFITTING OF BUILDING

INTRODUCTION OF RETROFITTING BUILDING

- The aftermath of an earthquake maneifests great devastation due to unpredicted Seismic motion striking extensive damage to innumerable buildings varying degree i.e. either full or partial or slight.
- This damage to structures in its turn causes irreparable loss of life with a large number of casualites.
- As a result frightened occupants may refuse to enter the building unless assured of the safety of the building from future earthquakes.
- It has been observed that majority of such earthquake damaged buildings may be safely resued, if they converted into seisemically resistance turctures by employing a few retrofitting measures.
- This proves to be a better option catering to the economic consideration an Immidiate shelter problems rather than replacement of buildings.
- Moreover it has often been seen that retrofitting of buildings is generally more Economical as compared to demolition and reconstruction even in case of severe structural damage.
- Therefore, seismic retrofitting of building structures is one of the most important aspects for mitigating seismic hazards especially in earthquake-prone countries.
- Varios terms are associated to retrofotting with a marginal differnce like Repair, strengethening, retrofitting, rehabilitation, reconstruction etc. but there Is no consensus on them.
- The need of seismic retrofitting of building arises under to circumstances:

1) Earthquake damaged buildings

2) Earthquake vulnerable buildings that have not yet experienced severe Earthquakes.

The problems faced by a structural engineer in retrofitting earthquake damaged Building

A. Lack of standards for methods of retrofitting;
B. Effectiveness of retrofitting techniques since there is a considerable Dearth of experience and data on retrofitted structures;
C. Absence of consensus on appropriate methods for the wide range of Parameters like type of structures, condition of materials, type of damage, Amount of damage, location of damage significance of damage, condition Under which a damaged element can be retrofitted etc.

CONCEPT OF VARIOUS TERMS ASSOCIATED WITH RETROFITTING

- In the recent past several devastating earthquakes around the world have demonstrated the lacunae in proper detailing of building structures and eventually the poorly detailed structures have become the victim of distresses of different kinds.
- During the post-disaster mitigation stage, a survey is required to investigate the conditions of the distressed building. Because of the vast variety of the building structures, the development of a general rule for retrofitting measure is rather difficult and to a large extent each structure must be approached as a strengthening problem on its own merits.
- It is necessary to take a decision whether to demolish a distressed structure or to restore the same for effective load carrying system. Many a times, the level of distress is such that with minimum restoration measure the building structure can be brought back to its normalcy and in such situation, restoration or retrofitting is preferred.
- It is known that certain types of building structures and a few specific components of these have repeatedly failed in earthquakes and are prime candidates for renovation and strengthening. Some of these are:

1. Buildings with irregular configurations such as those with abrupt changes in stiffness, large floor openings, very large floor heights etc.

(i) Buildings or structures on sites prone to liquefaction.

(ii) Buildings with walls of un-reinforced masonry, which tend to crack and crumble under severe ground motions.

(iii) Building with lack of ties between walls and floors or roofs.

(iv) Buildings with non-ductile concrete frames, where shear failure at beam-column joints and column failures are common.

(v) Concrete buildings in which insufficient lengths of bar anchorage are used.

(vi) Concrete buildings with flat-slab framing, which can be severely affected by large storey drifts.

The largest class of buildings in need of seismic upgrade is un-reinforced masonry buildings. These structures account for the majority of non-residential buildings and have certain problems in common. These buildings are commonly marred with scars after a string of powerful ground excitations.

The retrofitting of building structures involve improving its performance in earthquakes through one or more o

- increasing its strength and / or stiffness;
- increasing its ductility;
- reducing the input seismic loads.

The beginning of a typical renovation resembles a medical checkup of a first-time patient. An investigation of existing conditions is intended to determine the state of the building's health to establish a diagnosis and to arrive at a prognosis. A structure can be investigated in a variety of ways, depending on the type of structure, its apparent condition and whether the original design drawings are available. Multilevel approach to structural assessment of buildings is needed for a proper retrofit measure. The first level is a preliminary assessment that includes review of existing construction documents, site inspection, preliminary analysis of the structure and arrival at the preliminary conclusions and recommendations. Depending on the results of this stage, a second level, involving a more detailed assessment that deals with the same items in much more detail, may or may not be required. The steps necessary are:

(i) Reviewing existing construction documents
(ii) Field investigations
(iii) Probing and exploratory demolition
(iv) Testing materials
(v) Analyzing existing framing
(vi) Making an evaluation
(vii) Preparing a report of condition assessment

Material testing methods that involve removal and destruction of a portion of the member to determine its properties are called destructive testing. Nondestructive testing does not alter the members' properties or affect the service of the structure.

Residential retrofit

For detailed information concerning retrofit of certain types common wood frame structures OT exceeding two stories, see). For specific "permit ready" details as recommended by a public agency for simple low-rise construction.

Wood frame structure

Predominantly residential/dwelling in North America consisted of wood-frame structure. Wood is one of the best materials for anti-seismic construction since it is of low mass and is relatively less brittle than masonry. It is easy to work with and very cheap compared to other odern material as steel and reinforced concrete. This is only resistant if the structure is properly connected to its foundation and has adequate shear resistance, in modern construction obtained by well connected surfacing of panels with plywood or oriented strand board in combination with exterior stucco. Steel strapping and sheet forms are also used to connect elements securely.

Retrofit methods in older wood frame structures may consist of the following, and other methods not described here.

- The lowest plate rails of walls are bolted to a continuous foundation, or held down with rigid metal clips bolted to the foundation.
- Selected vertical elements, especially at wall junctures and window and door openings are attached securely to the sill plate.
- In two story buildings using "western" style construction (walls are progressively erected upon the lower story's upper diaphragm, unlike "eastern" *balloon framing*), the upper walls are connected to the lower walls with tension elements. In some cases, connections may be extended vertically to include retention of certain roof elements.
- Low *cripple walls* are made shear resistant by adding plywood at the corners, and by securing corners from opening with metal strapping or fixtures.
- Vertical posts may be restrained from jumping off of their footings.

Wooden framing is efficient when combined with masonry, if the structure is properly designed. In Turkey, the traditional houses (Baghdadi) are made with this technology. In El Salvador wood and bamboo are used for residential construction.

Reinforced and unreinforced masonry

In many parts of developing countries such as Pakistan, Iran and China, unreinforced or in some cases reinforced masonry is the predominantly form of structures for rural residential and dwelling. Masonry was also a common construction form in the early part of the 20th century, which implies that a substantial number of these at-risk masonry structures would have significant heritage value. Masonry walls that are not reinforced are especially hazardous. Such structures may be more appropriate for replacement than retrofit, but if the walls are the principal load bearing elements in structures of modest size they may be appropriately reinforced. It is especially important that floor and ceiling beams be securely attached to the walls. Additional vertical supports in the form of steel or reinforced concrete may be added.

In the western United States, much of what is seen as masonry is actually brick or stone veneer. Current construction rules dictate the amount of *tie–back* required, which consist of metal straps secured to vertical structural elements. These straps extend into mortar courses, securing the veneer to the primary structure.

Older structures may not secure this sufficiently for seismic safety. A weakly secured veneer in a house interior (sometimes used to face a fireplace from floor to ceiling) can be especially dangerous to occupants. Older masonry chimneys are also dangerous if they have substantial vertical extension above the roof.

These are prone to breakage at the roofline and may fall into the house in a single large piece. For retrofit, additional supports may be added or it may be better to simply remove the extension and replace it with lighter materials, with special piping replacing the flue tile and a wood structure replacing the masonry. This may be matched against existing brickwork by using very thin veneer (similar to a tile, but with the appearance of a brick).

RETROFITTING OF BUILDING STRUCTURES DAMAGED DUE TO EARTHQUAKE

In the recent past several devastating earthquakes around the world have demonstrated the lacunae in proper detailing of building structures and eventually the poorly detailed structures have become the victim of distresses of different kinds.

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The largest class of buildings in need of seismic upgrade is un-reinforced masonry buildings. These structures account for the majority of non-residential buildings and have certain problems in common. These buildings are commonly marred with scars after a string of powerful ground excitations.

The retrofitting of building structures involves improving its performance in earthquakes through one or more of: (i) increasing its strength and / or stiffness; (ii) increasing its ductility; (iii) reducing the input seismic loads.

The beginning of a typical renovation resembles a medical checkup of a first-time patient. An investigation of existing conditions is intended to determine the state of the building's health to establish a diagnosis and to arrive at a prognosis. A structure can be investigated in a variety of ways, depending on the type of structure, its apparent condition and whether the original design drawings are available. Multilevel approach to structural assessment of buildings is needed for a proper retrofit measure. The first level is a preliminary assessment that includes review of existing construction documents, site inspection, preliminary analysis of the structure and arrival at the preliminary conclusions and recommendations. Depending on the results of this stage, a second level, involving a more detailed assessment that deals with the same items in much more detail, may or may not be required. The steps necessary are:

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NONDESTRUCTIVE TESTING OF CONCRETE

Existing concrete can be tested by the following nondestructive methods:

(i) Visual inspection
(ii) Rebound hammer test
(iii) Hammer strike

(iv) Impact echo test

(v) Ultrasonic pulse velocity test

(vi) Pull off test

(vii) Cover meters & rebar locators

DESTRUCTIVE TESTING OF CONCRETE

The existing structures are required to damage to the extent of taking out samples. The following are the tests carried out by destructive process

(i) Taking cores & compression testing

(ii) Petro graphic analysis

(iii) Rapid soluble chloride test

(iv) Tension test of reinforcing bars

NONDESTRUCTIVE TESTING OF STRUCTURAL STEEL

Existing steel structural elements can be tested by the following nondestructive methods to determine the condition of steel members and their connections:

(i) Visual

(ii) Ultrasonic testing

(iii) Radiography

(iv) Magnetic particle test

(v) Liquid penetrate test

(vi) Hardness

2. **Seismic retrofitting** is the modification of existing structures to make them more resistant to seismic activity, ground motion, or soil failure due to earthquakes. With better

understanding of seismic demand on structures and with our recent experiences with large earthquakes near urban centers, the need of seismic retrofitting is well acknowledged. Prior to the introduction of modern seismic codes in the late 1960s for developed countries (US, Japan etc.) and late 1970s for many other parts of the world (Turkey, China etc.),[1]

3. many structures were designed without adequate detailing and reinforcement for seismic protection. In view of the imminent problem, various research work has been carried out. Furthermore, state-of-the-art technical guidelines for seismic assessment, retrofit and rehabilitation have been published around the world - such as the ASCE-SEI 41 [2] and the New Zealand Society for Earthquake Engineering (NZSEE)'s guidelines.[3]

4. The retrofit techniques outlined here are also applicable for other natural hazards such as tropical cyclones, tornadoes, and severe winds from thunderstorms. Whilst current practice of seismic it is similarly essential to reduce the hazards and losses from non-structural elements. It is also important to keep in mind that there is no such thing as an earthquake-proof structure, although seismic performance can be greatly enhanced through proper initial design or subsequent modifications.

SEISMIC RETROFITTING OF REINFORCED CONCRETE BUILDINGS USING TRADITIONAL AND INNOVATIVE TECHNIQUES

- The seismic retrofitting of reinforced concrete buildings not designed to withstand seismic action is considered. After briefly introducing how seismic action is described for design purposes, methods for

  Assessing the seismic vulnerability of existing buildings are presented.
- The traditional methods of seismic retrofitting are reviewed and their weak points are identified. Modern methods and philosophies of

  Seismic retrofitting, including base isolation and energy dissipation devices,

  Are reviewed.

- The presentation is illustrated by case studies of actual buildings where traditional and innovative retrofitting methods have been applied.
- Seismic retrofitting of constructions vulnerable to earthquakes is a current problem of great political and social relevance. Most of the Italian building stock is vulnerable to seismic action even if located in areas that have long been considered of high seismic hazard.
- During the past thirty years moderate to severe earthquakes have occurred in Italy at intervals of 5 to 10 years. Such events have clearly shown the vulnerability of the building stock in particular and of the built environment in general.
- The seismic hazard in the areas, where those earthquakes have occurred, has been known for a long time because of similar events that occurred in the past.
- It is therefore legitimate to ask why constructions vulnerable to earthquakes exist if people and institutions knew of the seismic hazard. Several causes may have contributed to the ration of such a situation. These are associated to historical events, fading memory, reed, avarice, poverty and ignorance. Among historical events particularly relevant are wars, pandemics, and natural disasters which may limit, in a significant way, the available resources of a country.
- In such circumstances there is a tendency to build with poor materials and without too much attention to good construction techniques and safety margins.

- A situation of this kind occurred in Italy and in Japan after the Second World War and imilarsituations have occurred in Italy many times in the past.
- In such a situation it is possible that the phenomenon of fading memory occurs and past emprise is easily erased.
- In Italy commercial profits often result from the employment of poor material and workmanship rather than of the optimal utilization of the production factors. The depressing situation of poor quality control and material acceptance also falls into this framework, which, in most cases, results only in paperwork devoid of substantive value. Marginal propensity to expenditure sometimes ensures that even the owner prefers a

low quality product to save resources for more immediate needs.

- Among causes arising from ignorance there may be both an inadequate knowledge of the seismic hazard and design errors due to insufficient knowledge of the earthquake problem; also the inability to correctly model the structural response to the seismic action.

Techniques

Common seismic retrofitting techniques fall into several categories:

One of many "earthquake bolts" found throughout period houses in the city of Charleston subsequent to the Charleston earthquake of 1886. They could be tightened and loosened to support the house without having to otherwise demolish the house due to instability. The bolts were directly loosely connected to the supporting frame of the house.

External post-tensioning

The use of external post-tensioning for new structural systems has been developed in the past decade. Under the PRESS (Precast Seismic Structural Systems), a large-scale U.S./Japan joint research program, unbounded post-tensioning high strength steel tendons have been used to achieve a moment-resisting system that has self-centering capacity. An extension of the same idea for seismic retrofitting has been experimentally tested for seismic retrofit of California bridges under a Caltrans research project and for seismic retrofit of non-ductile reinforced concrete frames. Pre-stressing can increase the capacity of structural elements such as beam, column and beam-column joints. It should be noted that external pre-stressing has been used for structural upgrade for gravity/live loading.

Active control system

Very tall buildings ("skyscrapers"), when built using modern lightweight materials, might sway uncomfortably (but not dangerously) in certain wind conditions. A solution to this problem is to include at some upper story a large mass, constrained, but free to move within a limited range, and moving on some sort of bearing system such as an air cushion or hydraulic film. Hydraulic pistons, powered by electric pumps and accumulators, are actively driven to counter the wind forces and natural resonances. These may also, if properly designed, be effective in controlling excessive motion - with or without applied power - in an earthquake. In general, though, modern steel frame high rise buildings are not as subject to dangerous motion as are medium rise (eight to ten story) buildings, as the resonant period of a tall and massive building is longer than the approximately one second shocks applied by an earthquake.

Adhoc addition of structural support/reinforcement

The most common form of seismic retrofit to lower buildings is adding strength to the existing structure to resist seismic forces. The strengthening may be limited to connections between existing building elements or it may involve adding primary resisting elements such as walls or frames, particularly in the lower stories.

Connections between buildings and their expansion additions

Frequently, building additions will not be strongly connected to the existing structure, but simply placed adjacent to it, with only minor continuity in flooring, siding, and roofing. As a result, the addition may have a different resonant period than the original structure, and they may easily detach from one another. The relative motion will then cause the two parts to collide, causing severe structural damage. Proper construction will tie the two building components rigidly together so that they behave as a single mass or employ dampers to expend the energy from relative motion, with appropriate allowance for this motion.

Exterior reinforcement of building

Exterior concrete columns

Historic buildings, made of unreinforced masonry, may have culturally important interior detailing or murals that should not be disturbed. In this case, the solution may be to add a number of steel, reinforced concrete, or poststressed concrete columns to the exterior. Careful attention must be paid to the connections with other members such as footings, top plates, and roof trusses.

Infill shear trusses

Shown here is an exterior shear reinforcement of a conventional reinforced concrete dormitory building. In this case, there was sufficient vertical strength in the building columns and sufficient shear strength in the lower stories that only limited shear reinforcement was required to make it earthquake resistant for this location near the Hayward fault

Massive exterior structure

In other circumstances, far greater reinforcement is required. In the structure shown at right — a parking garage over shops — the placement, detailing, and painting of the reinforcement becomes itself an architectural embellishment

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(iii) Radiography

(iv) Magnetic particle test

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(vi) Hardness

DESTRUCTIVE TEST OF STRUCTURAL STEEL

Common destructive tests of structural steel are:

(i) Chemical test

(ii) Bend test

(iii) Tension test

(iv) Compressio0n test

(v) Chary, Erode and drop weight impact test

(vi) Fatigue test

Masonry is one of the oldest and most common construction materials. A typical masonry Wall assembly consists of brick, block or stone units bonded together by mortar. It can also include horizontal and vertical reinforcing, embedded anchors, plaster and insulation.

NONDESTRUCTIVE TESTING OF MASONRY

(i) Visual inspection

(ii) Surface hardness

(iii) Stress wave technique

(iv) Petro graphic examination

DESTRUCTIVE TESTING OF MASONRY

(i) Compressive strength test

(ii) Modulus of elasticity

(iii) Petro graphic analysis

(iv) Moisture content test

RETROFITTING MEASURES

For retrofitting, there are no direct design guidelines, no codes, no standards and no practices for strengthening technology. The solutions adopted are generally based on successful prior practice. A few retrofitting measures are presented herein:

Renovating Steel framed buildings:

The need to reinforce existing steel beams by welding additional steel angles, channels or bars to act compositely with the original sections is quite common. Often this could be the only way to increase the load carrying capacity of the framing. Fig.1 shows the various ways of reinforcing the existing beams. While reinforcing the existing beam in the process indicated in the figure, the existing connection must be examined to ensure that they can carry the increased loading on the reinforced beam.

Like beams, existing columns can also be reinforced by welding cover plates or other sections. Columns need to have cover plates on both the flanges. Symmetrically placed reinforcing members may reduce the overall slenderness ratio of the combined section and make possible higher allowable stress in compression than existed originally. It is desirable to remove as much load from columns as possible before welding. For multistory structures, the effort of shoring several floors may not be cost effective and reinforcing them under stress may be inevitable.

Another peculiarity of reinforcing steel columns is a frequent need to fix deteriorated column bases such as that shown in Fig.2. The basic approach for this kind of repair is to shore the column, remove all the deteriorated material to sound material and weld or bolt the reinforcing to the column, designing the connection for the full load minus the load to be carried by the column, if any can be justified.

Another method of improving the load capacity of an existing steel beam is to make it act compositely with the concrete floor it carries. In composite construction, the slab becomes a part of the beam. In renovation, composite action can help substantially strengthen the existing beams and increase their stiffness

Renovating Concrete structural elements:

It is often easier and quicker to add structural steel rather than concrete members because new Concrete beams would require formwork and shoring and are difficult to build with the slab in place. To be effective, steel beams have to maintain deformational compatibility with the concrete beams they are intended to help. The load will be distributed among the new and existing beams in accordance with their relative rigidities (EI). Adding a steel channel on each side of an existing concrete beam is a common solution that allows the channels to be attached to the existing concrete columns. To share the load, the three beams can be interconnected by through bolting.

A different solution is shown in Fig.3b, where flexible steel channels fastened to the existing concrete only at the ends are used. The intention is to relieve the existing concrete member of some of the load by introducing upward forces into it. This is accomplished by deflecting the beams downward a predetermined amount, by jacking them, or by wedging the space between the underside of the slab and the beams.

In some cases, addition of steel beam may not be feasible from aesthetic considerations or something else. The strengthening can be done with concrete section enlargement. This procedure involves unloading the existing beam as much as possible, roughening its surface to remove contaminants and to improve the bond, and placing new reinforced concrete or shortcrete around the existing beam. Proper surface preparation and interconnection is critical to making the system function as a composite whole and to prevent de-lamination under load. The new and old existing concrete sections can be tied together by stirrups placed in horizontally drilled holes in the web of the existing beam (Fig.4a), by short dowels placed in drilled-in adhesive anchors (Fig.4b) or if strengthening is accompanied by a new floor overlay, by enveloping the existing beam.

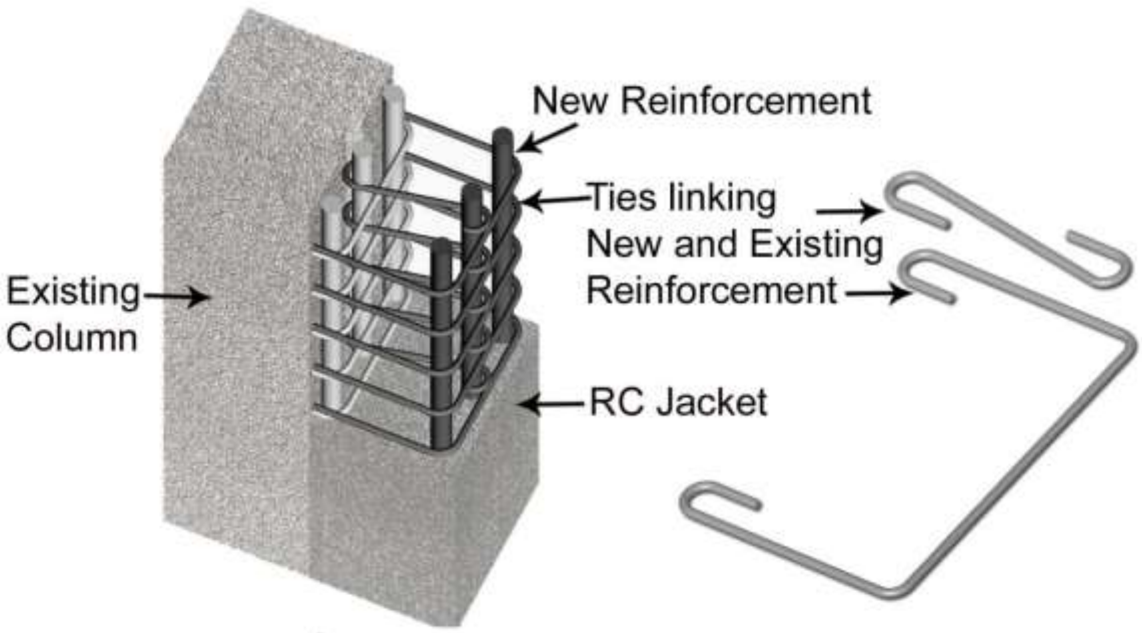
If the existing beam lacks positive moment capacity, it can be reinforced in place by adding structural steel tension plates or built-up members bolted to the beam. The welded U-bracket shown can be used if substantial additional steel area is needed. However, this is a passive design and the new steel does not become effective until the concrete deforms under some additional load.

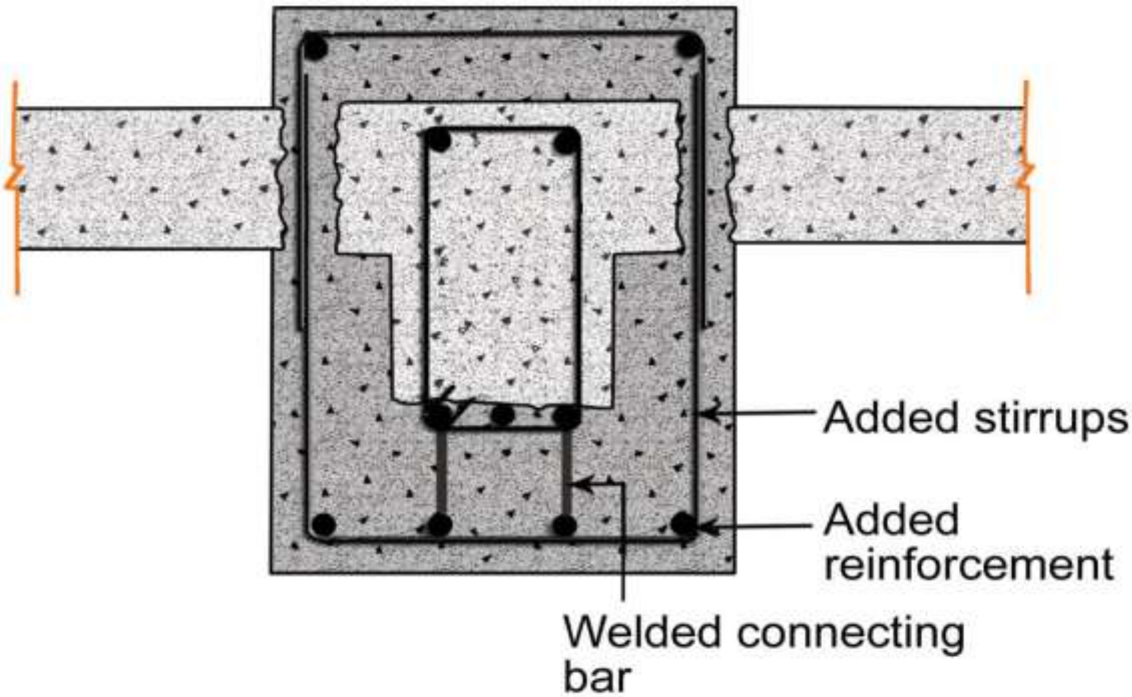
Plates of fiber reinforced plastics (FRP) can be used instead of steel plates. The advantage with FRP plate is the avoidance of corrosion problem, which is a problem for steel plates. FRP plates are most popular in the retrofitting of bridges as FRP is resistant to corrosion caused by acids, alkalis and salts. Both glass fiber reinforced plastic (GFRP) and carbon fiber reinforced plastic (CFRP) are used for retrofitting purposes.

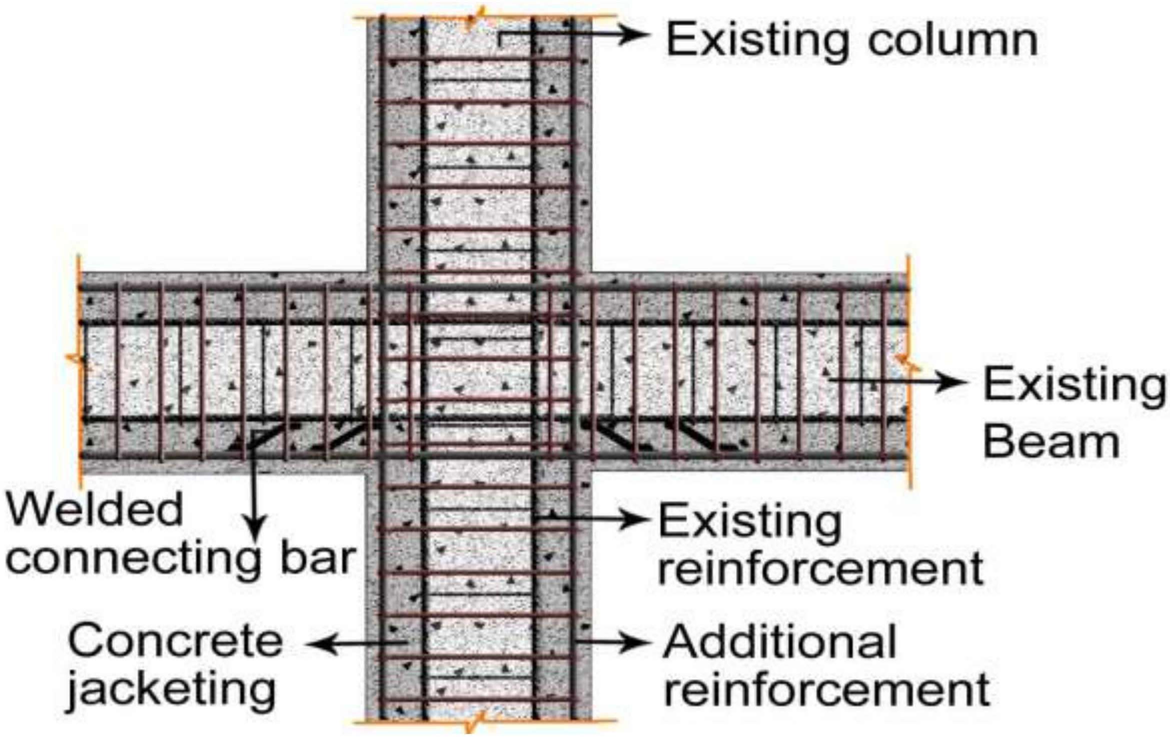
Large number of cracks of various sizes is generated in the concrete structures due to earthquake. There are three basic methods of crack repair: to ‘glue’ the cracked concrete back together by epoxy injection or grouting, to ‘stitch’ the cracked concrete with dowels or to enlarge the crack and ‘caulk’ it with a flexible or semi rigid sealant.

Jacketing, pinning, stitching, strapping etc. are some of the methods of retrofitting distressed structural elements. Depending on the types of distress and the importance of the structure an appropriate type of retrofitting technique is adopted.

Heritage structures need special sensitivity in retrofitting so as not to disfigure their appearnce and many such buildings have been successfully strengthened in different countries. Heritage buildings are often protected by statutory requirements, which make them difficult to deal with.




Added stirrups
Added reinforcement
Welded connecting bar


Existing column
Existing Beam
Welded connecting bar
Existing reinforcement
Concrete jacketing
Additional reinforcement

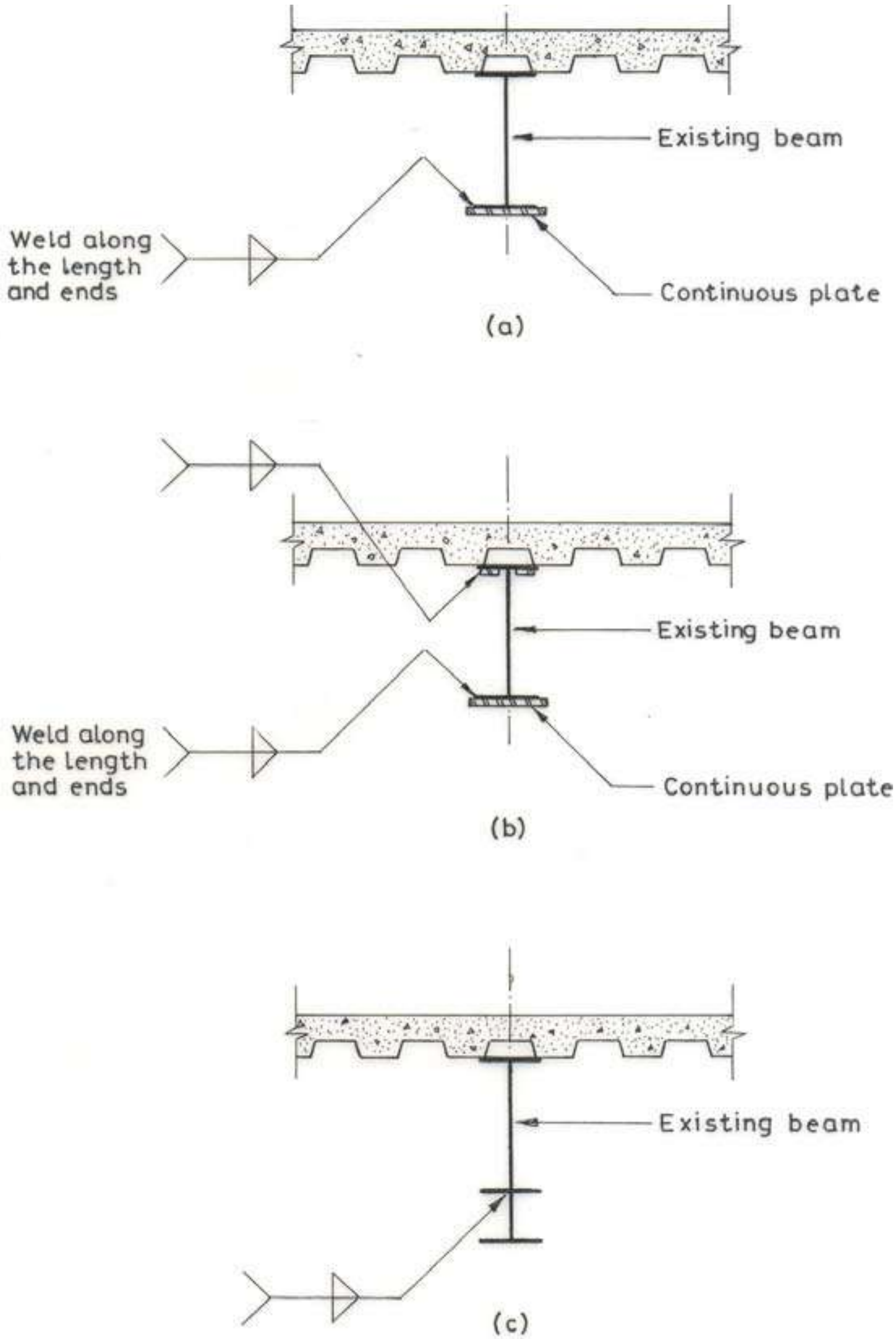


Fig.1 Reinforcing existing steel beams

Fig. 2 Deteriorated Column Base

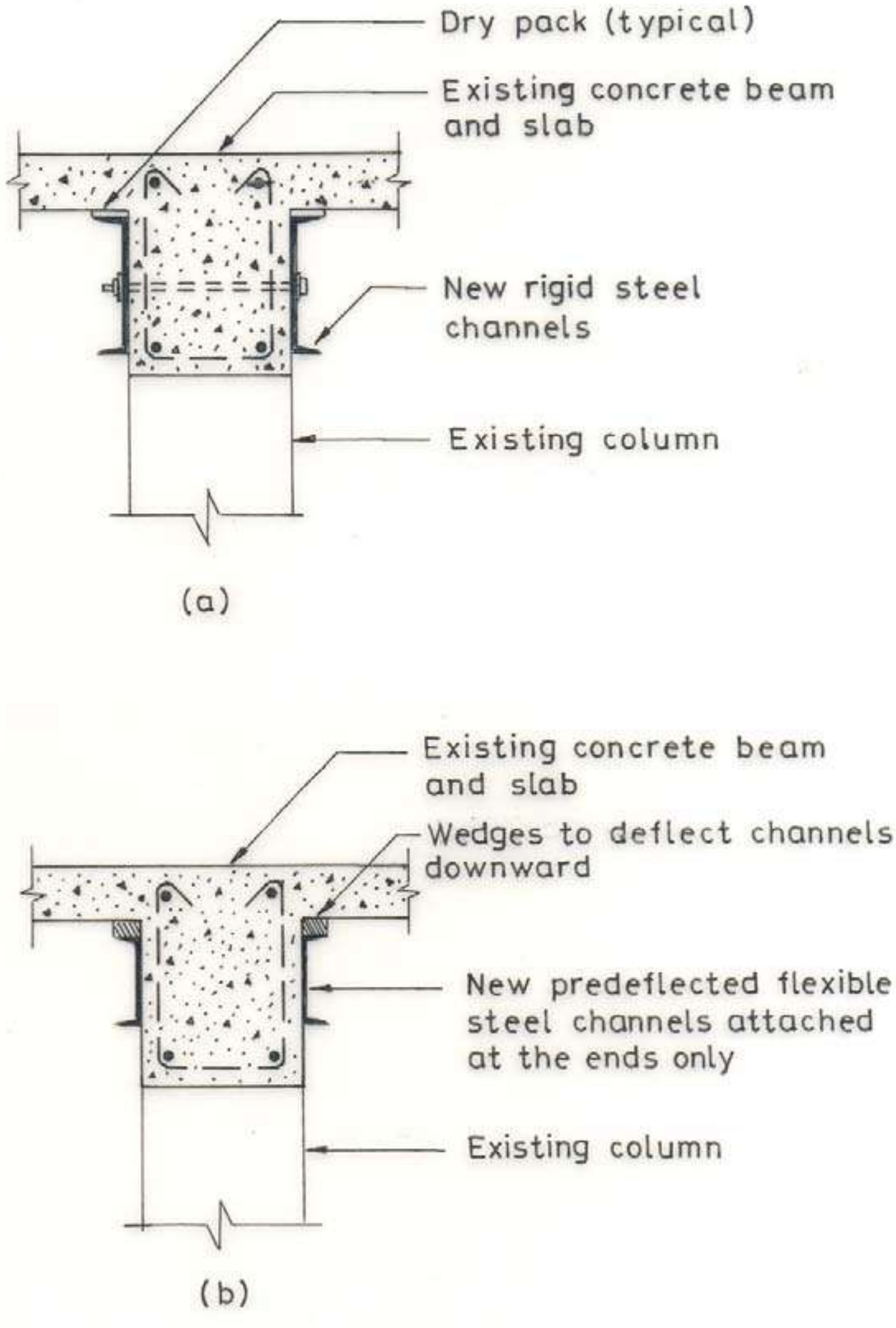


Fig. 3 Adding steel beams on each side of an existing concrete beam

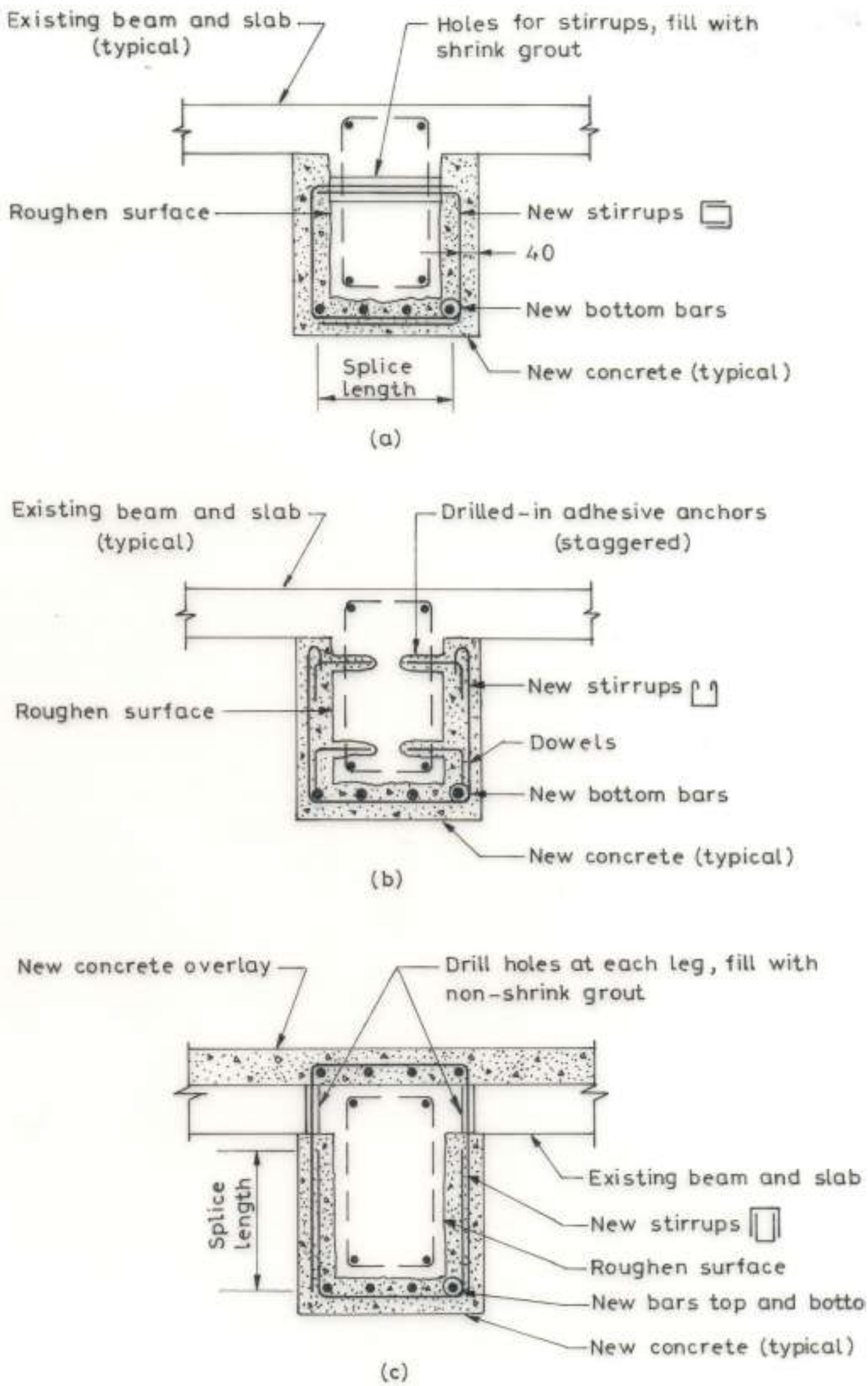


Fig. 4 Enlarging a section of an existing concrete beam

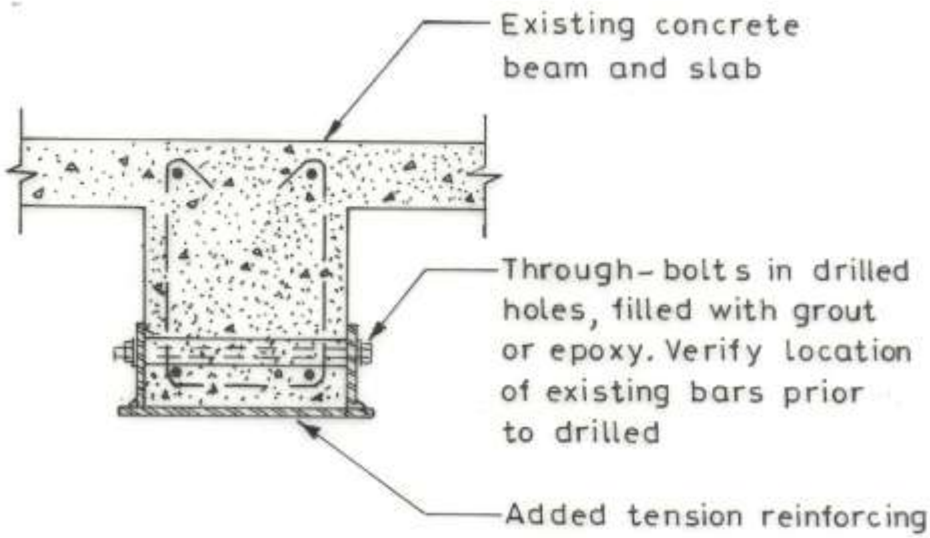


Fig. 5 Addition of steel member to improve positive moment capacity

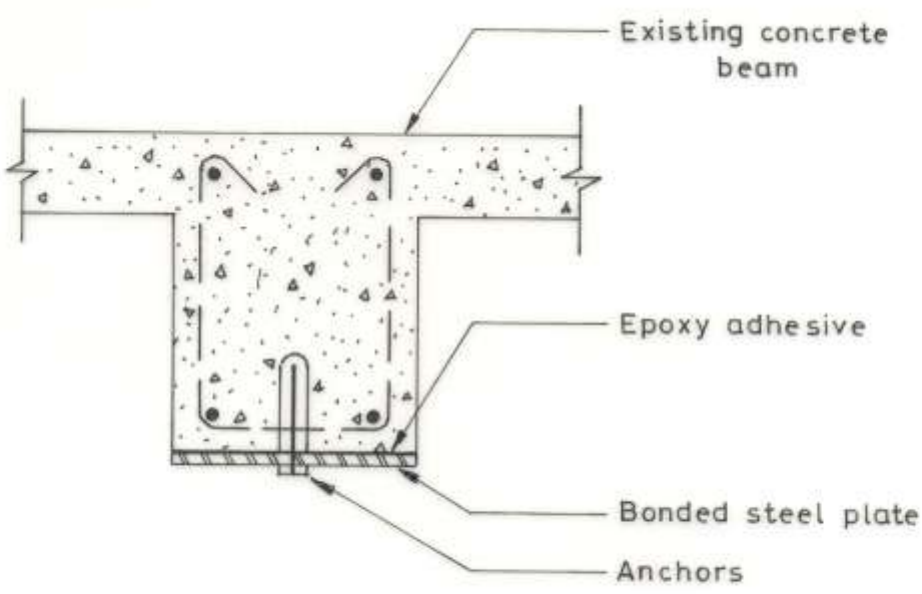


Fig. 6 (a)

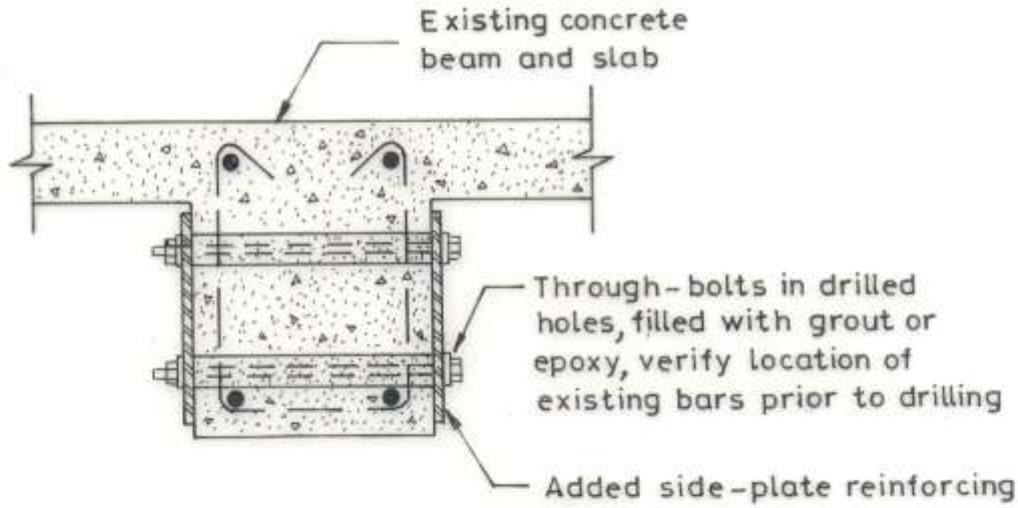

Existing concrete
beam and slab
Through-bolts in drilled
holes, filled with grout or
epoxy, verify location of
existing bars prior to drilling
Added side-plate reinforcing

Fig. 6(b)

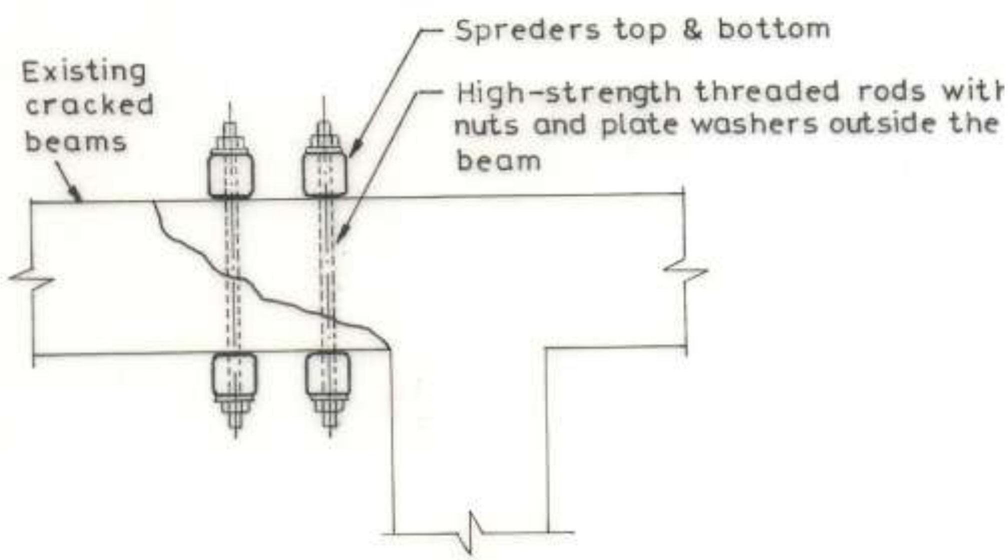

Spreders top & bottom
Existing
cracked
beams
High-strength threaded rods with
nuts and plate washers outside the
beam

Fig. 6(c)

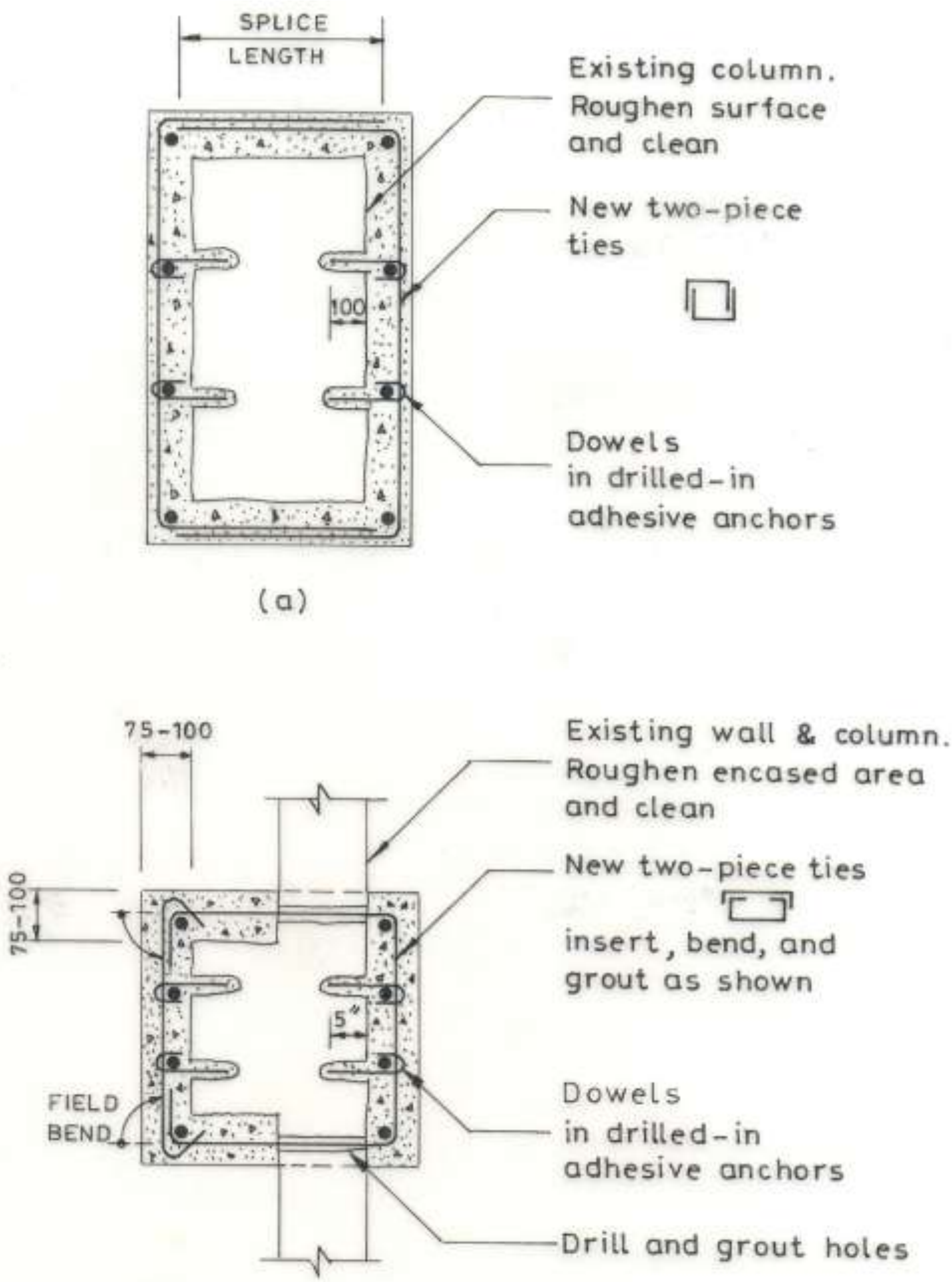


Fig.7 Retrofitting concrete columns

SEISMIC RETROFIT OF HISTO RIC BUILDING STRUCTURES

- Buildings with historic values are regional cultural assets worth preserving. The design technologies and building materials and methods that went into the original construction of these buildings are often drastically different from their contemporary counterparts, their structural renovation or retrofit brings forth many technical challenges to the design professional.
- This paper provides a general survey of the technical issues pertaining to the seismic retrofit of historic buildings, and explores various design procedures and construction methods for that purpose, including innovative technologies such as post tensioning, seismic isolation, composite wraps, etc.
- Special attention is given to the typical structural attributes of historic structures in terms of their structural stiffness, strength and ductility, how these parameters changed over the years, reliable methodologies for evaluating these primary structural attributes, and associated design implications for structural retrofit or hazard mitigations.
- Much of the discussion is based on a combination of the perspective provisions in building codes and alternative performance based approaches to meet the equilibrium, strain compatibility, and energy dissipation criteria, while a considerable weight is given to factors that influence preserving non-structural elements of historic value. A brief summary on cost implications is also provided.

Overview

- Buildings with historic value are regional cultural assets worth preserving. At times, they also represent a potential source of revenue and stimulus for the economical revitalization of their neighborhoods. The factors used to classify a building as historic may vary in different countries and cultures, so obviously not every aged building falls into historical or monumental category

- A building is historic if it is at least 50 years old, and is listed in or potentially eligible for the National Register of Historic Places and/or a state or local register as an individual structure, or as a contributing structure in a district.
- In prevailing practice, older structures are demolished and replaced by modern buildings due to economical and performance reasons, unless they can be claimed historic.
- The retrofit process is a general term that may consist of a variety of treatments, including: preservation, rehabilitation, restoration and reconstruction. Preservation is defined as the process of applying measures to sustain the existing form, integrity, and materials of a historic property.
- Rehabilitation refers to the process of creating new application for a property through repair, alterations and additions while preserving those features which convey its historical, cultural, or architectural values. Restoration is the process of accurately restoring a property as it existed at a particular period of time.
- Reconstruction is described as the act of replicating a property at a specific period of time. Selecting the appropriate treatment strategy is a great challenge involved in the retrofit process and must be determined individually for each project.

- Depending on project objectives, preservation and renovation of historic buildings may involve an array of diverse technical considerations, such as fire life safety, geotechnical hazards and remedies, weathering and water infiltration, structural performance under earthquake and wind loads, etc.

- Since the design methodology and building materials and methods that went into the original construction of these buildings are often drastically different from their contemporary counterparts, their structural renovation or retrofit brings forth many technical challenges to A/E design professionals.

Evolution of building materials

- Building materials have evolved gradually throughout the construction history, and the pace of the evolution is accelerated throughout the past century.
- Advancements in material engineering and metallurgy, invention of plastics and fiber reinforced composites, and innovations in production and treatment of existing building materials are some of the major causes of old and contemporary building material differences.
- Improvements in conventional building materials used both in historic and contemporary structures are described as:

Masonry, stone, and adobe buildings

- Bearing wall buildings were the dominant type of structures till late years of nineteenth century, when they were replaced by steel frame skeleton as the typical structural form in large buildings. In modern construction, masonry buildings are limited to certain building types and special locations.
- Natural stone has not changed, while adobe or bricks have slightly evolved to stronger, more durable building materials with consistent shapes and sizes. Design and construction techniques for masonry buildings are improved by using stronger mortar, and reinforcements to provide more resistance and continuity.
- Application of concrete filled blocks is also a major improvement in building masonry structures.

Wood and timber

- Wood, as a natural building material, has not been subjected to any major change, but modern technology provides strength grading methods, wooden panel products, preservation treatment process and wood protection.

Concrete

- Concrete has been subjected to significant evolution during twentieth century. Improved ingredients, quality control, preparing, and casting process offered stronger and more durable concretes. Improvements in concrete technology, application of additives, plasticizers, and improved cements provide light weight, high strength, high workability, shrinkage compensation, low porosity, and fiber reinforced types of concrete.

Hot-rolled reinforcing steel

- Reinforcing steel has evolved considerably regarding the material properties and shape. Reinforcement bars initially had square cross-sections, high carbon content, and smooth surface, where new ribbed, reinforcement bars with limited carbon content provide more ductility and stronger bond between the steel reinforcement and concrete.

Structural steel

- Overall strength of structural steel was improved within past century (See Table 1). Section dimensions and properties of steel shapes have also been changed and a number of shapes are considered obsolete and they are no longer produced. Difference in strength, ductility and weldability must be considered in the retrofit design process.

Practice and design concepts

- Building codes have been constantly updated in past decades on the basis of various lessons learned from previous failures (especially earthquake related failures).
- Advances in computer programs and hardware have drastically changed the way we do structural analysis and design. As a rule, newer provisions tend to prescribe better continuity for seismic loadings, provide more redundancy in structural system, and they exploit inelastic structural capacities to absorb and dissipate earthquake loads.

- Such contemporary code requirements and engineering knowledge base were not available to designers and builders at the time historic buildings were typically designed and constructed without detailed assessment of the probabilistic magnitude of loading (especially load cases related to wind or earthquake) or clear knowledge on structural behavior.

- Design methodologies were also quite limited in past days, when engineers were required to perform hand calculations with numerous estimations in the process. Older design concepts required that working stresses remain within elastic limits.

- Higher engineering approximations accompanied by older design concepts, resulted in over-designed structural members which do not necessarily improve seismic behavior, but they usually add to dead loads.

- Older design concepts mostly focused on the effects of gravity loads and they did not dedicate enough attention to provide adequate lateral resistance and ductility. Most of historic buildings provide limited ductility and continuity, especially when subjected to seismic loading. Unreinforced bearing walls provide limited resistance against lateral loading and a high potential of discontinuity at corners or connection to the roof.

- It is very common to notice historic reinforced concrete building with discontinued flexural reinforcements, no transverse reinforcement in beam-column joint zones and minimal confinement in columns. Retrofit process requires local modification of components, minimizing structural irregularities (in mass and stiffness), structural stiffening, structural strengthening, mass reduction and seismic isolation to improve the structural performance and comply with current building codes (i.e. FEMA356, IBC2003, UBC1997).

- Performance objectives used for historic retrofit are similar to general objectives used in the performance based engineering context, but with extra constraints to preserving the historic fabric along with the structure itself.

- In most cases, the façade and fixtures are of historic value and preserving them requires limiting deformation imposed by seismic loads. Limiting deformations is in contrast with the newer design philosophies that exploit the structural ductility to reduce the required strength. In seismic retrofit of historic buildings both the global strength and stiffness must be increased to minimize the deformation and damage to the historic fabric.

Challenges of retrofitting historic fabric

- Minimizing noise, disturbance, and damage to the surrounding buildings and providing temporary shoring and support are typical challenges involved in most retrofit projects. Depending on the extends of retrofitting, assessed risk, technical limitations, structural historic value, and economical constraints, the preferred retrofit strategies are studied and prioritized to preserve the authenticity of historic fabrication and minimize removal of architectural material:

No penetration of building envelope

- The process does not require any destructive procedure so the historic fabrication remains untouched (e.g. composite wraps or chemical treatment).
- This approach is only applicable to very limited cases since structural components are mostly either embedded in or covered by the finishing.

Penetration without breakage

- The structural component subjected to retrofitting is accessible, and the retrofit process only requires drilling holes (e.g. micro piles, epoxy injection, post tensioning).

Breakage with repair

- In many cases, some destructive procedures are required to access the structural component or to perform retrofit process (e.g. fixing and improving welded connections or installation of base-isolators).

Replace

- In cases structural components cannot be improved to meet retrofitting objectives or the damage or deterioration could not be repaired, components are replaced. Replacement process requires special attention to providing support for the rest of the building, isolating the component, and maintaining continuity.

Rebuild

- In cases a feasible retrofitting solution cannot be found, the historic building is reconstructed, partially or as a whole. This option imposes greater economical burden and the loss of authenticity may have impacts on historic and cultural values.
- Typically rehabilitation of historic buildings requires new structural members and preservation of historic fabric is accomplished by hiding the new structural members or by exposing them as admittedly new elements in the building's history. Often, the exposure of new structural members is preferred because alterations of this kind are reversible and they could conceivably be undone at a future time with no loss of historic fabric to the building.

Innovative technologies for historic preservation

- Modern materials and equipment provide many retrofit options to improve the behavior of structural system, global strength, stiffness or mitigate the seismic hazards. Some of the commonly used techniques in retrofitting are listed below:

Post tensioning

- Post tensioning is considered one of the potentially efficient retrofit option ns for reinforced concrete or masonry buildings, providing strength and ductility to the overall structure with minimal intrusion. Masonry has a relatively large compressive strength but only a low tensile strength.
- It is most effective in carrying gravity loads. However, in-plane shear and out of- plane lateral loads induce high levels of tensile stress also. Commonly, these induced tensile

stresses exceed the compressive stresses and reinforcing (commonly with steel members) must be added to provide the necessary strength and ductility.

- The level of compressive stresses can be significantly raised by post-tensioning the reinforcing steel and the more brittle tensile failures avoided.
- Basically, a core hole is placed down through the masonry wall and a high-strength steel rod (or tendon) is inserted. The bottom of the rod is anchored in the floor or foundation. A jack is then used at the top of the wall to place high levels of tensile force in the rod.

Base isolation

- Base isolators are used to decouple the building response from the ground motion and in the event of a major earthquake, base isolation will greatly reduce structural and architectural damage, mostly by shifting the structure natural period
- The two basic types of isolation systems that have been employed are elastomeric bearings (using natural rubber or neoprene) and the sliders (Teflon and stainless steel).

- Structural members and of the entire construction. Also, changes in service conditions, often made arbitrarily, may lead to substantial changes in the structural behavior resulting in a degradation of the structural response to the expected loading conditions.

- Tthe basis of what has been presented so far, it is not surprising that in areas long known to be subject to the seismic hazard it is not infrequent to find constructions vulnerable to earthquakes.

- These constructions need to be retrofitted to allow them to withstand the effects of the earthquake ground motion expected at the site considered. In the following sections some procedures used for the evaluation of the seismic resistance and vulnerability of reinforced concrete buildings will be described together with traditional and innovative techniques of seismic retrofitting of the same structures.

- The paper ends with a description of the seismic retrofitting of two reinforced concrete residential buildings in the village of Soaring, near Syracuse, in Sicily.

- The buildings belong to the Institute Autonomy Case Popularity (IACP) of Syracuse. As will be clear from following arguments the aim of the paper is not to discuss in depth the state-of the-art of seismic retrofitting, but rather to give a general overview.

- The aim is also to focus on a few specific procedures which may improve the state-of-the-art practice for the evaluation of seismic vulnerability of existing reinforced concrete buildings and for their seismic retrofitting by means of innovative techniques such as base isolation and energy dissipation.

SEISMIC ACTION

- Seismic vulnerability is not an absolute concept but is strongly related to the event being considered. The same construction may not be vulnerable to one class of earthquakes and yet be vulnerable to another.
- Therefore, before attempting a seismic vulnerability evaluation of a given construction, the seismic action that will affect that construction must be fully specified.

- All seismic codes specify the seismic action by means of one or more design spectra. These are a synthetic and quantitative representation of the seismic action which, besides depending on the characteristics of the ground motion, depends on some intrinsic characteristics of the structure such as the fundamental mode of vibration and its energy dissipation capacity.

- The elastic design spectrum depends on the vibration periods of the structure and on the available damping. In Figure 1 the elastic spectrum of Euro code 8 (CEN, 1998) is drawn for three different values of damping. A new draft of Euro code 8 (CEN, 2003) became available in 2003, but is not being used here because some of the Euro code 8 material relevant to the present work is still questionable and not generally accepted.

- The value of the spectral pseudo-acceleration, corresponding to a vanishing small period, corresponds to the peak ground acceleration (PGA). In fact, for $T = 0$ the structure is rigid and, therefore, subject to the same acceleration as the ground.

- This acceleration, called the maximum effective ground acceleration or PGA, depends directly on the seismic hazard at the construction site and acts as the anchoring acceleration of the spectrum. This value is generally prescribed by seismic codes as a function of the seismic hazard at the construction site.

- Furthermore, four regions may be identified for the elastic spectrum, each defined by a lower and upper period. In the first region, $(0 \leq T \leq TB)$, the spectral ordinates increase linearly with the period; in the second $(TB \leq T \leq TC)$, these are independent of the period; in the third $(TC \leq T \leq TD)$, the spectral ordinates decrease rapidly with the period, that is with the reciprocal of the period T according to Euro code 8; and finally in the fourth region $(T \geq TD)$, they decrease even more rapidly, with the reciprocal of the period squared according to Euro code 8. More details on the elastic design spectrum may be found in the seismic codes (CEN, 1998), in specialized publications and in the treatises on dynamics of structures and seismic engineering (Chopra, 2001; Clough and Pension, 1993).

- The separation periods *TB*, *TC*, *TD* depend on seismological factors and on local site conditions. For instance Euro code 8 specifies them as a function of three subsoil classes: A (firm soil), B (medium soil), C (soft soil)

- In traditional seismic design the energy dissipation capacity of the structure deriving from plastic Deformations is generally considered. Including the inelastic resources of a structure allows for a Considerable reduction of the spectral ordinates in the design spectrum. This reduction generally depends on the available ductility and on the vibration period. Euro code 8 considers.

- This reduction is mainly dependent on a factor related to ductility and it is described as structure behavior factor or simply structure factor. Typical values of the structure factor *q* may fall in the range 1 to 5 for reinforced concrete structures (CEN, 1998). As may be seen from Figure the use of the inelastic resources of a structure allows for a considerable reduction in the spectral ordinates and therefore in the design strength.

INNOVATIVE APPROACHES TO SEISMIC RETROFITTING

- The main innovative methods of seismic retrofitting may be grouped into the following classes:
  - Stiffness reduction
  - Ductility increase
  - Damage controlled structures
  - Composite materials
  - Any suitable combination of the above methods
  - Active control.

  - For equal mass the 'stiffness reduction' produces a period elongation and a consequent reduction of the seismic action and therefore of the seismic strength demand. The stiffness reduction may be achieved by the principle of springs in series whereby the equivalent stiffness of two springs in series is smaller than either of the single springs as shown in Figure 6.
  - In general it may be assumed that base isolation is a special case of the stiffness reduction approach. Although very effective, this method must be used with a pinch of salt.
  - Too low a stiffness may result in large displacements, especially inter-story drifts, which may conflict with the functioning of the building and cause damage to non-structural components.
  - Therefore deformability checks are always a must. Instances in which this method may not be effective are the cases of long period structures or of stiff structures on soft soils. In the first case the advantages gained by a reasonable increase in period may be negligible;

- In the second case the stiffness reduction may be counterproductive by leading to an increase of spectral ordinates. An application of the 'stiffness reduction method' will be shown in some detail in a further section.
- A 'ductility increase' may be achieved locally by confinement of reinforced concrete flexural as well as compressed structural members. Although this method has a long history, it may now be applied easily using new materials such as fiber reinforced polymers (FRP). These materials are distinguishable by the type of fiber and the most common are denoted by CRP, GRP, ARP, indicating respectively reinforcement with carbon (C), glass (G) and Aramaic (A) fibers.

EVALUATION OF SEISMIC RESISTANCE AND VULNERABILITY

1. Definition of SDOF Equivalent Systems

- The seismic resistance and, consequently, vulnerability of reinforced concrete constructions may be evaluated by means of a procedure proposed within some documents of the Federal Emergency Management Agency (BSSC, 1997a, 1997b). These documents have been subsequently upgraded to prestandard level, FEMA 356 (BSSC, 2000); however, while document FEMA 356 (BSSC, 2000) is intended to supersede document FEMA 273 (BSSC, 1997a), document FEMA 274 (BSSC, 1997b) remains the basic commentary also to the pre-standard. The FEMA procedure has been modified by some research work carried out at the University of Catania (Olivet et al., 2001).
- The results that will be obtained within the present paper use the modified procedure. An elastic-plastic incremental analysis of the structure under the seismic action is a necessary prerequisite.
- The seismic action is defined in terms of the forces corresponding to the first few modes of vibration of the structure or in terms of the pseudo static forces prescribed by seismic regulations.
- The results of the incremental analysis come in the form of storey force-displacement curves commonly known as push-over curves. On the basis of these curves a single-degree-of-freedom (SDOF) equivalent system is defined.

- Before describing the procedure in some detail it is appropriate to notice that the procedure may be used for the evaluation of the seismic resistance of existing buildings as well as that of new ones (in the design stage). As such the procedure may also be used for the evaluation of the effectiveness of seismic retrofitting projects shows a reinforced concrete building before and after retrofitting according to the stiffness and resistance increment concept. Besides demonstrating the type of retrofitting system which has been used in this case, the pictures illustrate the complexity of the structure on which the incremental analysis must be performed.
- Further details on the procedure used for the design of the retrofitting systems for a class of buildings of the type shown in Figure 10 may be found in Olivet and Decennia (1998). For the sake of clarity it should be noted that the building in Figure 10 was retrofitted in the early nineties, before the FEMA procedures became available and before the subsequent studies by the senior author and his co-workers.
- The building is shown here to provide an example of seismic retrofitting by increase of resistance and stiffness and to illustrate the complexity of systems on which push-over analyses must be performed.
- This is the reason why the push-over analysis described below was not performed on this building but on a four storey building described in detail in Olivet et al.
- The storey force-displacement (push-over) curves have been constructed using commercial and research computer programs.
- The use of commercial programs has been undertaken in order to ensure a quick transfer of the research results to the seismic engineering profession. More details and the relevant literature may be found in Olivet et al. (2001).
- The analyses have been performed along two orthogonal directions roughly corresponding to the axes of symmetry of the plan of the building; in fact the chosen directions were those of the corresponding first modes of vibration of the building.
- The analyses have been performed, using approximations described in detail in Olivet et al. (2001), on 3D models of the buildings considered.
- The results of the push-over analyses. Here the storey force-displacement Curves are shown for each of the storey of the building considered, together with the work performed by the storey forces as functions of the base shear of the building. Because the floors are considered

as rigid for in-plane strains and the building is nearly symmetrical, any floor point may be considered in the construction of the storey force-displacement curves.

- For each step of the incremental (pushover) elastic-plastic analysis the storey forces are known and the corresponding floor displacements are calculated.
- The analysis is stopped when the first plastic hinge breaks, on the assumption that this leads to a stress redistribution and subsequent plastic hinge failures as in a chain reaction. The displacement of the SDOF equivalent system is evaluated on the basis of the work equivalence.
- The equivalence is established in incremental as well as in global terms. The shaded area in Figure is the sum of the shaded areas.
- The work equivalence defined above is not limited to symmetrical buildings with in-plane rigid floor slabs, but can be established for any structural system.
- A mathematical equivalence for general multi-degree-of-freedom (MDOF) systems may be found in Olivet.

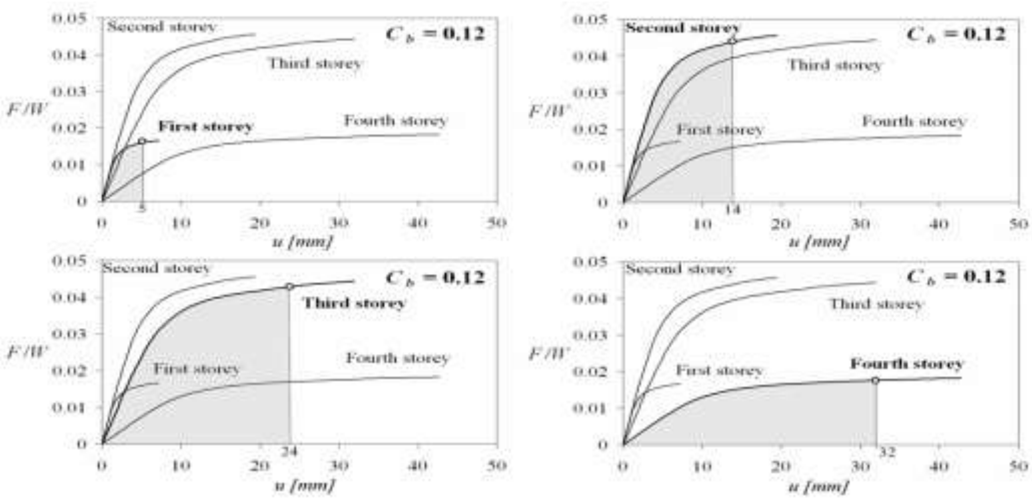


Fig. Storey force-displacement (push-over) curves for the construction of the equivalents OF system

- The graph in defines the equivalent SDOF system of the building in terms of base shear and the corresponding displacement as established by using work equivalence. Perhaps it may be worth noticing at this point that the base shear coefficient Cb = 0.12. is an arbitrarily chosen value of the ratio between the base shear force and the weight of the building in the interval 0

$\leq Cb \leq Cb,c$ with Cb,c = 0.125 being the collapse base shear coefficient. Given the weight W of the building, to each Cb there corresponds a specific base shear force and specific storey forces as Cb = 0.12.

- Obviously the equivalent SDOF system should be defined for the two principal directions of the building. Therefore at least two equivalent SDOF systems of the form shown in Figure must be evaluated for each building according to the previously outlined procedure.

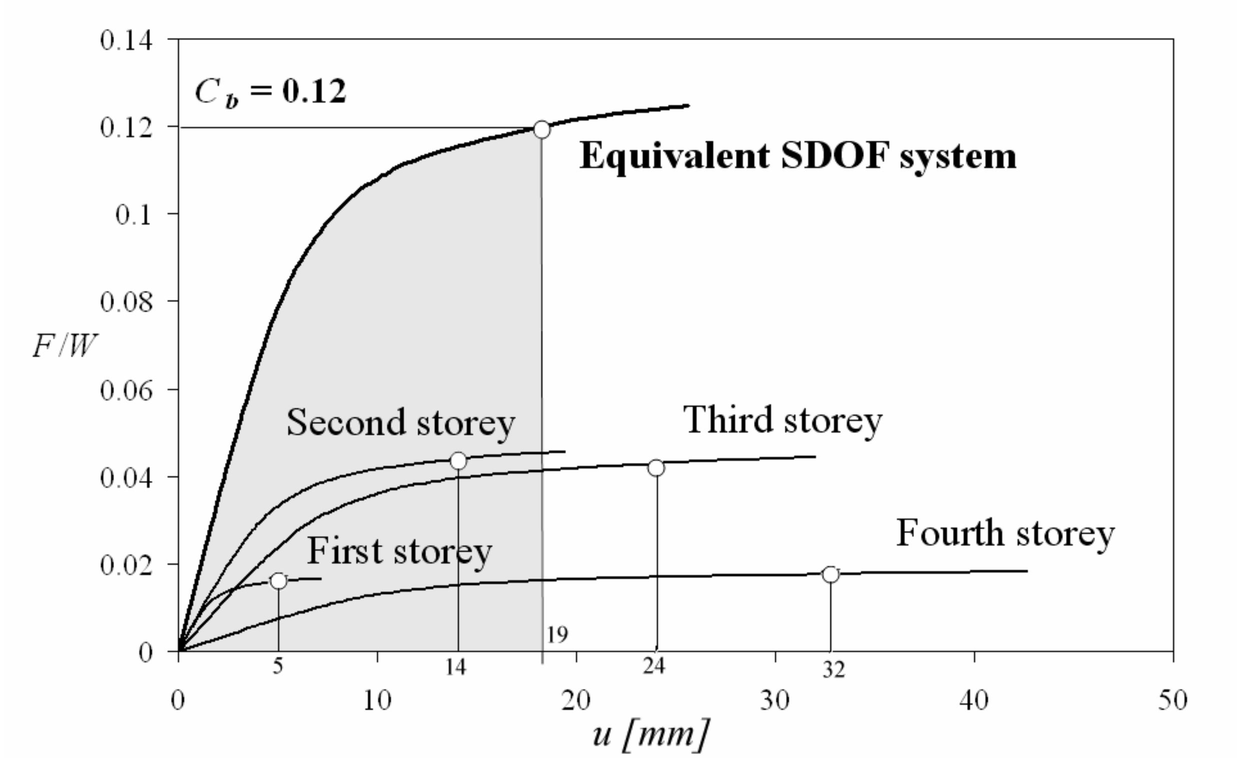


Fig. Evaluation of the equivalent SDOF system on the basis of the storey forcedisplacement (push-over) curves

2. Seismic Resistance in Terms of Effective Peak Ground Acceleration (PGA)

- The characteristics of the force-displacement curve of the equivalent SDOF system are used to establish the seismic resistance of the building in terms of the effective PGA. For this some preliminary considerations relating to the design spectrum and to the interaction between the spectral ordinates and the PGA are required. This interaction is clearly.

- The first operation that must be performed on the equivalent SDOF system is the substitution of the continuous non-linear curve with a bi-linear one. Of the two linear segments, the first one is considered elastic while the second is elastic-plastic with hardening. The substitution is achieved by using the work equivalence and the condition that the second linear segment should be tangent to the actual curve at point *C*.

- In this way three characteristic points are identified, *Y'*, *Y* and *C*, two of which belong to the original system, that is *Y'* and *C*, and two belong to the new one, that is *Y* and *C*. Point *C* corresponds, at the same time, to the maximum base shear and to the corresponding equivalent deformation.

- The value *Cb,c* of the base shear coefficient corresponding to point *C* defines one of the unknown parameters in Equation (4). Point *Y*, corresponding to the vertex of the bi-linear system, is related to the definition of the effective elastic stiffness of the equivalent SDOF system. As shown in Figure 14, this may be evaluated as:

2.1 A Note on the Spectral Shape Function f (Teff ,S,ζ)

- The spectral shape function *f* (*Teff* ,*S*,ζ) appearing in Equation (9) has different expressions for the elastic and the inelastic design spectra in the range of periods $T \geq TC$ according to Eurocode 8 (CEN, 1998). In the same range of periods, relevant literature suggests using the same spectral shape for elastic and inelastic behavior.
- For the sake of simplicity the elastic spectral shape of Eurocode 8 has been used in the following numerical applications.

Seismic Resistance and Vulnerability

- The seismic resistance defined in terms of effective PGA by means of Equation (9) represents a measure of the maximum ground motion that a building can withstand at the threshold of collapse.

- It is interesting to compare this value with the corresponding value that the seismic regulation prescribes for the construction site. By denoting the latter with *ag*,*c* the relative seismic resistance may be defined as:

Application to Buildings in the Village of Soaring

- The procedure described in the previous sub-sections has been applied to two almost identical buildings in the village of Soaring of the province of Syracuse.
- The relevant results of the push-overanalyzes and other properties of the building. The meaning of the symbols is the same as introduced in Sub-section 2 above.
- The ductility ratio was evaluated with the refined method proposed by Marletta and Olivet and is somewhat smaller than the value predicted by Equation.

- According to present seismic regulations the building site is in an area of medium seismic hazard and local site conditions may be classified as Type A according to Eurocode 8. For research purposes the analysis has also been repeated for the zones of low and high seismicity and for the soil conditions of Types B and C, thus covering the complete spectrum of seismicity and site conditions covered.

- The results in terms of relative seismic resistance. The regional seismic hazard is specified in Italy in terms of effective PGA as follows: PGA = 0.35g for the areas of high seismic hazard, PGA = 0.25g for the areas of medium seismic hazard, and PGA = 0.15g for the areas of low seismic hazard. Just recently a fourth area of minimal seismic hazard has been proposed with PGA = 0.05g.

- From an examination of Table 2 it appears that the building would be vulnerable to the design earthquake independently of local soil conditions if located in a zone of high or medium seismic hazard.

- If the building were situated in a zone of low seismic hazard it would be able to withstand the design seismic action only on firm soil, that is Type A soil condition. From the same table it may be seen that the transverse direction is the one with less seismic resistance. The same results in terms of seismic vulnerability.

- Provides a vulnerability index for the building with reference to the design earthquake. The ‘0’ value shows that the building is not vulnerable while ‘1’ (100%) indicates that the building has no seismic resistance. Intermediate situations have an obvious meaning. Data on seismic over-resistance has not been shown because it was available in just one case.

SEISMIC RETROFITTING BY STIFFNESS REDUCTION

- The buildings owned by IACP of Syracuse in the village of Solar no , the seismic vulnerability of which has been evaluated in the previous section, have been considered for seismic retrofitting by means of stiffness reduction, and one of the original buildings .
- The IACP buildings in Solar no seemed to invite the designer to retrofit by stiffness reduction. In fact, by looking at the original foundations, it was clear how easy it would be to support the building, to cut the short columns between the foundation and the first floor slab, and to insert the devices that would ensure the stiffness reduction. Also, a detailed geological study confirmed the rocky nature of the foundation soil, thus excluding high long period components in the expected ground motion and confirming Class A soil condition according. The devices for stiffness reduction as used in the present case.

- As may be seen from Figure 18 the building is supported by 12 elastomeric bearings low-friction bearings. The elastomeric bearings, commonly known as seismic isolators, besides to the stiffness reduction, introduce also a significant energy dissipation capacity.

- The low friction bearings, which could rightly be called seismic isolators, have the function of transmitting vertical loads to the foundation, while limiting any possible horizontal action to the bare minimum. Preliminary investigations on materials and structural members have shown an excessive deformability at local and global levels, so much so that the structure would not have been safe under gravity and seismic loads even after the stiffness reduction.

For this reason a further retrofitting action has been undertaken to reduce this high deformability. The proposed design.

Fig. 15 Building owned by IACP of Syracuse in the village of Solarino

Fig. 16 Foundations of an IACP building in Solar no

- The building stiffening by thin reinforced concrete walls, of thickness 15 cm, allows not only for an improvement of the vertical load carrying capacity and for the deformability limitation, but also for a much better behaviour of the stiffness reduction mechanism and of the entire building.

- It should be noticed here that the inserted reinforced concrete walls stiffen and strengthen only the superstructure while the overall stiffness is essentially determined by the base isolation system.
- Therefore the overall system, while attracting lower seismic forces, is better suited to withstand the seismic forces affecting the superstructure with well-controlled inter-storey drifts

Resistance and Vulnerability

- The resistance analysis conducted with the previously outlined procedure has produced the results.
- The force distribution used in the push-over analyses was again that corresponding to the first vibration mode in the direction considered, which in displacement terms is practically constant with all displacement occurring at the level of the base isolation bearings and the building essentially behaving as a rigid body.

- For the three classes of seismic hazard considered in Italy, the results based on soil conditions of Type A, that is the soil condition existing at the construction site and best suited for retrofitting by the base isolation, are shown.

- Besides the results referring to the building retrofitted with walls and isolators (denoted by SR+W, the symbols indicating stiffness reduction plus walls), those referring to the original building are given for comparison along with those of the hypothetical building strengthened by the presence of the walls.

- It is evident that the retrofitted building has an over-resistance for all the classes of seismic hazard, which is obviously decreasing as the level of seismic hazard increases.

- Overall the building strengthened only with thin walls would be safe only in the areas of low seismic hazard. The situation shown in Table 4 in terms of seismic resistance is reconsidered in terms of vulnerability and in terms of over-resistance.

- Major earthquakes have indicated that the seismic retrofit of existing buildings is necessary because the buildings may fail to satisfy the latest seismic design provisions. In this paper, a novel seismic retrofit plan with rocking walls and steel dampers for a multistory steel reinforced concrete frame is proposed.

- The following two aspects form the primary focus: 1) The possibility of weak story failure of the existing SRC frame should be eliminated.

- The difficulty suppressing the unintended weak story failure in frame structures is evident from building damages in historic and recent earthquakes, despite various implementations of the widely accepted "strong column-weak beam" concept in the seismic design of frame structures. Instead of a strength hierarchy between beams and columns, the effect of continuous columns on reducing the story drift concentration has been extensively examined for steel frames.

- These attempts may lead to an effective solution for suppressing the weak story mechanism in frames. 2) Damage to the existing frame should be minimized. The SRC frame presented herein was designed and constructed during the late 1970s before the major revision of the seismic code in Japan in 1981, which was mainly a consequence of the 1978 M7.1 Miyagiken-oki earthquake.

- As suggested by the damage observed in the M7.3 Kobe earthquake 1995, SRC frames designed and constructed in old days usually lack deformability to accommodate damages .

- The above concerns, a rocking wall system are developed to enhance the seismic performance of the existing SRC frame. Rocking walls are global vertical components that are strong and stiff and have sufficient rotating capacity at the bottom.

- They are responsible for controlling the deformation pattern along the height of the structure to reduce the story drift concentration. Rocking walls need to be firmly connected to the rest of the structure to ensure that the lateral forces can be transmitted. Concentrated vertical deformation that forms when the structure deforms laterally.

- It's expected that most of the energy dissipations as well as damages will be concentrated in the energy dissipating devices to minimize damage to the rest of the structure. The advantages of rocking wall systems have been explored by Kumara et al.

- They pressed several precast concrete wall panels together with post-tension tendons to form a rocking wall. Marriot et al [12] introduced steel dampers at the bottom of the rocking wall to increase the energy dissipation capacity.

- The first applications of a rocking wall system were in a newly built 4-story office building and in the rehabilitation of an existing 6-story RC moment-resisting frame.

- The rocking wall system to be introduced in this paper differs from the previous studies in the following aspects: 1) the rocking interface between rocking walls and their foundations is replaced by explicit pin bearings to avoid unfavorable impact at both corners of the wall by placing steel dampers on both sides of the rocking wall, energy dissipation is distributed along the height of the rocking wall, rather than being concentrated at the bottom, which permits more energy dissipation devices to be used in the structural system to greatly increase the energy dissipation capacity

- The post-tensioning of the rocking walls is only responsible for increasing the crack strength of the rocking walls, rather than providing any self-centering capacity to the system.

- On the one hand, it is thought that the strength and stiffness of the rocking wall is much more important than its self-centering capacity, and on the other hand, anchoring the post-tension tendon on the wall instead of in the foundations considerably reduces the cost of strengthening the foundations.

Seismic Retrofit of G3 Building in India Tech

- The G3 Building is an 11-story steel reinforced concrete frame structure on the Suzukakedai campus of the Tokyo Institute of Technology.
- As mentioned above, it was designed and constructed before the major revision of the seismic code of Japan and it has already been occupied for more 30 years. As concluded by a recent seismic inspection, there is an urgent need to strengthen the structure, especially in its longitudinal direction.

Retrofit plan

- The north view of the retrofitted G3 Building. During the retrofit, the building remained occupied because most of the construction was done from outside the building.
- The structural plan of the G3 Building before and after retrofitting. There are several other multi-story concrete buildings on the same campus with similar configurations to that of the G3 Building.
- A common feather is that there are several slots along the perimeter of the building. This feather makes it easier to implement the rocking wall system. For the G3 Building, 6 pieces of post-tensioned concrete walls with pin bearings at the bottom were installed in the existing slots and firmly connected to the existing frame at each floor level by horizontal trusses. Shear steel dampers were installed in the gaps between the rocking walls and adjacent existing SRC columns as well as between the rocking walls and the added transverse walls at both ends. Main components of the rocking wall system.
- the post-tensioned concrete walls, the steel dampers, and the bottom pin bearings, are visible from outside the building; thus people can see them and appreciate the engineering solution. The seismic behavior of the retrofitted G3 building is different from that of a shear wall-frame structure and a moment-resisting frame. No recommendations for the seismic design of such a structural system are available yet.
- Nevertheless, several basic criteria regarding the expected seismic performance of the retrofitted structures are met. First, the post-tensioned rocking walls should remain elastic,

even if the structure is subjected to major earthquakes, such as the Level II earthquake in the design.

- In other words, the rocking walls should not yield or crack, which may significantly impair their stiffness. Second, the story drift ratio of the structure should remain below 1/200 during a major earthquake.
- This requirement is very strict compared with current seismic codes for reinforced concrete structures. However, it is believed necessary in the current case considering the fact that the existing SRC frame is built before 1981, and its deformability might be rather poor. Lastly, steel dampers at different levels of the structure should be proportioned such that the energy dissipation is as evenly distributed along the height of the building as possible.

- Bearing in mind these concepts, nonlinear time history analysis are carried out to determine the earthquake action on each part of the structure and to evaluate the seismic performance. In the following, these key components are described in detail.

of

Post tensioned concrete walls

- Because the rocking walls are responsible for controlling the deformation pattern of the structure, it is expected that their stiffness and strength can be retained even under a major earthquake. All 6 pieces of post-tensioned concrete walls have identical cross-sections with a width of 4300mm and a depth of 600mm.
- The total cross section area of the rocking walls at each story is about 50% to 61% of that of the existing SRC columns from the bottom to the top story. Concrete with a nominal compressive strength of 36MPa is used. Each rocking wall is pre-stressed by 6 units of post-tensioned tendons to increase its cracking strength. Each tendon unit comprises 30 strands of 12.7mm.
- The initial pre-stress for each rocking wall is 22500kN, and the corresponding control stress is about 68% of its nominal tensile strength. The resultant effective pre-stress is over 18000kN for each rocking wall.

Connections for rocking walls

- Rocking walls are connected to the foundation and the existing structure. Cast iron pin bearings are installed at the bottom of the rocking walls.

- Details and a photo of the completed bearing are shown in Figure 5. It was designed to resist large shear force while permitting the wall to rotate freely around its base.

- The cast iron bearing consists of two separated tooth-shaped pieces (the lower and the upper piece), which interlock with several teeth and a separated stopper in the middle to prevent displacement in the out-of-plane direction of the wall.

- The teeth in the lower piece are 20mm longer than those in the upper ones to create a small gap, and their tips are filleted to allow for rotations of the upper piece.

- Cast rocking wall Rocking iron NCN490 with nominal yield strength of no less than 325MPa was used for the bearings.

- It should also be noted that the rocking walls have little effect on the fundamental period and the maximum base sheer force of the existing structure.

- As a result, the foundation work for the rocking walls is not excessive, and the shear demand for the pin bearing is not very large.

- Rocking walls are connected to the existing structures by the horizontal trusses at each floor level in the slots of the existing structure behind the rocking walls.

- It can be seen in Figure 7 that the horizontal trusses are firmly connected to the existing structures by anchor bolts. Steel shear keys are used to connect the horizontal truss and the rocking wall to permit the rocking walls to rotate while transmitting the lateral force.

Shear steel damper

- Shear steel dampers are installed on both sides of the rocking walls. Low yield steel SLY225 with a nominal yield strength of 225MPa was used for the 6mm steel web of the damper, which functions as the energy dissipater and is constrained by transverse ribs with a spacing of 250 mm. The web height H was 312 mm for all the dampers, and the length L varied from

750 mm to 1500 mm. Figure 8 shows details and a photo of a completed steel damper with a web length of 1500 mm. The cyclic loading test of the steel dampers shows that the nominal strength of the damper can be satisfactorily retained up to 9% shear strain, which is about 58 times the yield shear strain of the damper.

- The nominal strength of the damper is calculated by multiplying the steel nominal shear strength (taken as and the cross section area of the web [15]. The deformation of 750 mm steel damper at the end of the test as well as its hysteresis loop is shown in Figure 9. Most of the earthquake input energy is expected to be dissipated by these dampers.

Seismic performance assessment

- Nonlinear time history analysis was carried out to assess the seismic performance of the structure before and after the retrofit. Two ground motion records, NGT-NS and JMA Kobe-NS, are used and their acceleration time histories are depicted.
- The peak ground accelerations (PGA) and peak ground velocities (PGV) are listed in Table 1. They generally represent a Level II earthquake ground motion in the seismic design practice in Japan, i.e. PGV=50cm/s.

- Two-dimensional member-by-member finite element models are built in ABAQUS 6.8. A fiber-based beam element is used to model the existing SRC frame, and user-defined uniaxial materials were used for concrete fibers and steel fibers.

- The behavior of the steel dampers was idealized as an elastic-perfectly plastic model, and the rocking walls were assumed to remain elastic through the analysis. The maximum story drift ratios of the structure before and after the retrofit under the above ground motions are shown in Figure 12. It is obvious that the deformation of the structure is significantly reduced and is below the 1/200 criteria under both ground motions after being retrofitted.

- Furthermore, the deformations in different stories are much more evenly distributed along the height of the structure, which indicates that the damage is spread throughout the structure so that excessive damage is not concentrated in a local part of the structure, which could cause premature failure of the whole structure

SEISMIC RETROFIT OF EXISTING BUILDINGS

- ***GENERAL* –** History has shown that light, wood-frame residential buildings with specific structural weaknesses in their original construction are susceptible to severe damage from earthquakes. The most common structural weaknesses are: 1) absence of proper connection between the exterior walls and the foundation (i.e., anchor bolts), 2) inadequate bracing of cripple walls between the foundation and first floor, and 3) discontinuous or inadequate foundations below the exterior walls. (Comoro & Levin, 1982) (Steenburgen, 1990) Unreinforced masonry chimneys and poorly reinforced or tied reinforced masonry chimneys are also a communed in the scope of this document, since reduction of chimney vulnerability through pointing of mortar and bracing is not typically considered cost-effective particularly if the risks to life can be controlled by other means. For example, ATC recommends adding plywood above the ceiling framing to reduce the chances of falling masonry from penetrating through the ceiling (ATC, 2002).
- Curtailing the occupancy and frequent use of property within the falling radius of chimneys is also an effective way of minimizing the risk of casualties. ATC recommends replacement of upper portions of damaged chimneys with light-framed construction rather than diagonal bracing.

Purpose –

- In contrast to other earthquake retrofit guidelines and codes, the provisions of this chapter are not strictly designed for life safety protection. These provisions are, in fact, expected to reduce property damage, reduce the number of uninhabitable dwellings after earthquakes and avoid the increased public assistance expenditures to repair damage and provide temporary housing (ABAG, 1999).

- These provisions do not guarantee that strengthened structures will not be damaged. However, it is anticipated that costly damage to the vulnerable parts of dwellings below the first floor will be greatly reduced.

- These requirements have not been established or calibrated using Performance Based Earthquake Engineering, so there is no intent to state or imply a performance objective or range particularly since these requirements don't address vulnerabilities that might exist above the first floor of dwellings. "Cripple walls retrofitted to these provisions are generally expected to meet a Life Safety performance objective" (CUREE, 2002). However, other parts of the building may exceed or perform less than this objective requires.

- Analysis and design of strengthening for other structural or nonstructural components not addressed by these provisions must be performed in accordance with Section 301.3 Alternative Design Procedures.

Scope –

- "Prescriptive" means these provisions apply to specific conditions and must be used in precisely the manner described. "Prescriptive" also means determined in advance, without the need for case-specific analysis or design.
- Through the use of these provisions and the accompanying details, the dwelling owner or contractor can develop plans without the services of a Design Professional (Civil Engineer, Structural Engineer or Architect). However, the use of other materials, proprietary systems or methods not shown by the figures and details within this Chapter, may require the services of a Design Professional.

- The provisions are intended to deal with specific earthquake weaknesses. Therefore, the work that is being done is structural in nature and requires submitting plans and obtaining a building permit. The weaknesses that these provisions address are listed in Section 303.

- It should be clearly understood that the application of these provisions is limited because the provisions are not suitable to use for strengthening hotels, motels or large multi-unit apartments since they are typically larger structures that require engineered retrofits by licensed design professionals.

- Also excluded are dwellings built with cripple walls with studs taller than 4 feet in any location, dwellings that have columns or poles embedded in the ground as their foundation system, and buildings exceeding 3 stories or any 3 story building with cripple wall studs exceeding 14 inches in height. Each of these types of buildings presents unique conditions that preclude the use of prescriptive criteria to strengthen them.

- The four feet cripple wall stud height limits the amount of overturning in braced crippled walls with lengths defined in Figure A3-10. When the height of the wall studs exceeds four feet, the owner will need to have the bracing designed by an engineer or architect. Conventional construction provisions in the IBC Section 2308.12.4 state that cripple walls with studs exceeding 14 inches in height are to be considered as first story walls for the purpose of determining bracing. However, the bracing layout requirements in these provisions are provided according to the number of stories above the cripple wall.

- Items 3 and 4 in the Exception to Section 301.2 refer to the cripple stud height. The parameter H, represented in Figure 3-7, is greater than the stud height and is not used in these provisions.

- The building official is allowed to also exclude other residential buildings that these provisions would otherwise apply to if they have vertical or horizontal irregularities or other features not considered by these prescriptive standards.

- Very few dwellings are rectangular in plan, and U- T- or L-shaped plans aren't necessarily problems in conventional construction with wood diaphragms. However, split-level dwellings, hillside dwellings, or structures where the upper level exterior walls are horizontally offset from the line of the lower story exterior walls may be determined by the building official to be beyond the scope of these provisions. It is recommended that the owner or contractor consult with the authority having jurisdiction before beginning work to determine if these provisions can be applied.

- Observations after past earthquakes have shown that cripple walls with substantially varying heights, as at sloping or stepped foundations, suffer more damage than cripple walls of constant height (SEAOSC, 2002). Most of the forces go to the shorter, stiffer portions of the cripple wall. For multi-story buildings with more than a 2 to 1 height ratio between the tallest and shortest cripple wall bracing panels, it is recommended that the sheathed panel lengths be engineered along the wall lines in question.

- The provisions of this Chapter are intended for uses in high seismic regions (Section 301.2). Still, for dwellings within 10 km of major active faults (Ss > 1.5) it is recommended that the panel lengths be engineered to account for potentially more severe ground motions in the design earthquake, since these provisions do not account for near source ground motions. Other structural weaknesses that may exist that are located above the first floor are also beyond the scope of this chapter.

- Situations that require analysis and design by a Design Professional or consultation with the Building Department prior to beginning any work include but are not limited to buildings with full-height stone veneer walls due to their added weight, and dwellings built on hillsides where cripple wall heights vary substantially. Also excluded from these provisions are buildings or portions of buildings constructed on concrete slabs-on-grade.

- These dwellings do not have cripple stud walls and typically would not lack bracing. These buildings may have wall anchorage deficiencies, and the provisions for wall anchorage of cripple wall buildings apply equally well to these structures. However, retrofit of these structures would require removal of wall finishes and may not be as cost-effective as retrofitting those buildings with crawl spaces.

- Also, while sliding of dwellings on slab-on-grade foundations has occasionally occurred in past earthquakes, it has not caused widespread economic and habitation losses in past earthquakes. Dwellings with stem walls (reinforced concrete or masonry foundation walls that project above the ground to the underside of the first floor framing) will experience substantial damage if the welling slides off the foundation, and anchorage of these structures should be considered as falling in the scope.

- A majority of dwellings constructed in California prior to 1950 were unanchored. The Uniform Building Code did not begin to specify anchorage until its 1946 Edition (SEAOC 1995), however most local governments did not uniformly adopt such model codes promptly after their publication until the late 1970’s. Furthermore, some dwellings were constructed with inadequate cripple wall bracing in the 1970s and even later particularly where model codes were not enforced. Dwellings often used horizontal wood siding or stucco as wall sheathing material.

- Owners of older dwellings should examine the exterior walls from within the crawl space under the first floor to determine if the sill plates are bolted to the foundation, the bolt size and spacing complies with the Building Code or Table 3-A and if exterior finishes are applied over wall sheathing materials with adequate strength such as plywood, OSB or diagonal sheathing. Most dwellings constructed after 1950 were anchored to their foundations.

- However, even in high seismic regions , cripple wall bracing now considered inadequate was in common use in the 1970’s and 1980’s.

Alternative Design Procedures

- Section 301.3 purposely omits the commonly used statement that the design must comply with all the requirements of the Building Code because complete code compliance is often not feasible with respect to existing buildings. For example, Design Professionals should not be expected to rigorously address issues such as the stiffness variations in existing flooring systems due to differences in the type or thickness of the flooring. It does, however, state that any strengthening designed by the Design Professional should at least be equivalent in terms of strength, deflection and capacity to that provided by the prescriptive methods. The Building Official is allowed to require Design Professionals to provide substantiating structural calculations or test data to confirm this equivalence.

- This 75 percent factor applied to Building Code design forces accounts in a general way for differences between current design criteria and the less conservative criteria that were likely in effect when the dwelling was originally designed and built. If owners prefer, they can elect to use a greater horizontal force in order to lessen potential damage from future earthquakes.

DEFINITIONS

- Throughout this chapter there are references to "the Building Code. " This term generally refers to the current edition of the International Building Code (IBC) or the jurisdiction's governing code. Before using this document to determine the amount of strengthening that may be required, consult with the local building department to confirm the appropriateness of using these definitions.
- The definitions provided in this chapter are for terms that are not defined in the governing Building Codes. Other terms defined in the Building Code also apply to this chapter but are not repeated.

STRUCTURAL WEAKNESSES

- This section provides criteria that allow owners AND CONTRACTORS to evaluate dwellings The Building Code provisions are essentially reproduced in the figures and tables of this chapter.
- The structural weaknesses listed in this section might not be the only weaknesses that will lead to structural damage when the building is subjected to earthquake forces.
- They are, however, the most common and most cost effective to strengthen and represent those that can be addressed by prescriptive, no engineered provisions.

Approved Foundation System.

- Some older dwellings do not have a foundation system. Instead, the wall sill plate, and much of the floor framing, is supported directly on the ground.

- When subjected to earthquake induced lateral and vertical forces, these structures can easily move because they are not anchored. Fungus, water, and insect damage are also common in unapproved foundations.
- This movement can result in a variety of structural and nonstructural damage including broken gas and utility lines that can lead to -fires. Further, these structures are highly susceptible to both fungus infection and insect infestation due to inadequate wood to earth separation. Wood deterioration caused by this inadequacy has significantly contributed to the damage resulting from earthquakes.

Pier Foundation System.

- Many dwellings have continuous cripple walls and foundations around their perimeter and wood posts on isolated concrete pad footings (called a “post and pier” system) under the interior of the dwelling. Such a system is not necessarily deficient.
- However, if a “post and pier” system forms the foundation for the dwelling perimeter walls, this is considered a structural weakness because of the lack of stiff walls below the first floor. T
- his deficient foundation system is found in some older dwellings, including Victorian era structures, and buildings in areas where soil moisture is high. The posts and floor framing members of this system are usually interconnected with simple toenailed connections with no bracing between the posts.
- This weakness is compounded when a lack of connection occurs between the posts and the small concrete pads which act as footings. Failures in this foundation system during earthquakes occur at the underside of the floor framing, and may lead to partial collapse of the structure.
- Providing diagonal bracing members between the posts does not solve the problem. Each post would then need to be adequately connected to a foundation system. Typically, the existing footing pads are too small to make the necessary connections.

- Therefore, simply providing bracing between the posts only moves the point of failure from the top of the post to the bottom of the post at the footing pad. For these buildings, bracing, anchorage and provision of additional footings may be required. Some post and pier type structures may be considered as historic buildings.
- Care needs to be taken when performing strengthening work so the historic nature of the building is not destroyed. Many jurisdictions have adopted specific requirements for historical buildings such as those in Chapter 10 of the IEBC. If a dwelling utilizes this type of system and might be considered an historic building, consult with the building department before beginning any strengthening work.

Non-Continuous Perimeter Foundation Systems*.*

- Another deficiency is found in dwellings that do not have a continuous perimeter foundation. However, there are many variations of partial foundations, and some do not represent a significant weakness.
- When applying these provisions to an existing building the intent is to reduce the potential for damage to habitable portions of structures.
- Therefore, the standards include two exceptions to the requirement for continuous perimeter foundations.

Unreinforced Masonry Perimeter Foundation.

- A perimeter foundation constructed of unreinforced masonry is assumed to lack the necessary strength to resist earthquake forces.
- These systems are common in many older dwellings built before codes were adopted in high seismic regions and may also exist in newer dwellings where codes have not been enforced. When subjected to earthquakes these systems are easily damaged, allowing the building to shift off foundations. Section 304.2.2 requires analysis of unreinforced masonry foundations by either an architect or an engineer.

Inadequate Sill Plate Anchorage*.*

- While sliding between an unanchored sill plate and the foundation can occur, it is actually one of the more rare sources of damage. However, it is still important that sill plate anchorage be present in order to complete the lateral force path.
- Bracing cripple walls without bolting the sill plate to the foundation simply moves the weak link to the interface of the sill and the foundation. Compliance with either Tables 3-A and B or the Building Code is acceptable.

Inadequate Cripple Wall Bracing.

- Past earthquakes have shown that the most common cause of major damage in dwellings is due to poorly braced cripple walls. (LA, 1994) In high seismic regions, the following cripple wall bracing methods are gypsum board, fiberboard, particleboard, lath and plaster, or gypsum sheathing boards are acceptable materials for bracing single story dwellings, and wood structural panels and diagonal wood sheathing are acceptable materials for bracing multi-story dwellings. For additional information and connection requirements for these materials.
- A common and very weak type of cripple wall can be found in older buildings constructed with horizontal, exterior wood siding.
- This type of siding, and it's nailing, is not adequate to resist earthquake forces associated with nearby moderate or major earthquakes. Recent earthquakes have also shown that "let-in" diagonal bracing does not adequately brace cripple walls.
- Let-in braces are usually nominal 1" thick and placed in a notch cut into the face of the stud. Let-in braces are no longer permitted as an acceptable bracing method in IBC for buildings located in regions where strong earthquakes are expected to occur.

- Let-in braces should not be confused with diagonal wood sheathing. Diagonal wood sheathing, which is acceptable by these provisions, is composed of individual boards nominally 1" thick, laid diagonally across the face of the stud wall.

- These boards are laid next to one another covering the entire width and length of the wall extending from the top plate to the sill plate. If the cripple walls are covered with diagonal sheathing, the wall is adequately braced, provided the boards are nailed to each stud they cross and to the top and bottom plates.

- Adequate nailing consists of three 8-penny nails at each stud and the ends. If the boards or the studs are split, or if the end nails are too close to the ends of the sheathing, this system can be deficient.

- The most effective cripple wall bracing system that significantly reduces the risk of damage is wood structural.

- If the dwelling has plywood sheathing as an exterior finish check for nails spaced no more than 6 inches apart along all the edges of each sheet. If adequate nailing is not present, comply with the nailing requirements of this Chapter.

- Exterior plywood siding with vertical grooves (referred to as T1-11) can have another serious deficiency. At the edges where two adjacent panels adjoin, each panel must be nailed to the wall stud with a separate row of nails.

- These sheets have “lips” so that they overlap at the joints.

- A common, improper construction practices providing only one row of nails through both sheets (at the overlap). This creates a weakness as the plywood thickness is only one-half of its normal thickness at the overlap, and only half the number of nails is provided. Such practice led to failures in the 1984 Morgan Hill (California) Earthquake. In all cases where nailing is exposed to the elements, it is recommended that hot-dip galvanized nails be used. A dwelling with existing Portland cement plaster (stucco) as the exterior finish might not have its cripple walls adequately braced by this material.

- Stucco has been a recognized bracing material for a number of years but it is only as good as the connection of the lath to the studs and plates. Many dwellings with stucco applied directly over the studs without plywood or diagonal sheathing under the stucco experienced serious damage in the 1994 Northridge Earthquake (LA, 1994)(NAHB, 1994). In high seismic regions, most often this failure was due to inadequate attachment of the lath to the bottom sill plate (LA, 1994). Through the years there have been various lath systems used for installing stucco. Stucco is normally applied in three coats (7/8 inch total thickness).

- When subjected to high loads it can fail in diagonal tension represented by diagonal cracks. To increase the tension capacity, stucco is reinforced with wire lath. This reinforcing does not keep the stucco from cracking but helps prevent cracks from opening. Consequently, existing stucco containing diagonal cracks must be carefully evaluated.

STRENGTHENING REQUIREMENTS

Scope –

- Use of materials, proprietary systems or methods not shown by the figures and details within this chapter requires the services of a Design Professional. Where a dwelling has an unusual or irregular configuration or unusual features, the services of an engineer or architects to design a strengthening program utilizing the alternate procedures of Section 301.3 is required.
- The Building Official may require a pre-design special inspection as described in A304.5 and C304.5 to determine which portions of the work require the services of a Design Professional. CUREE recommends a number of enhancements to these provisions that are currently under consideration by the SEAOC Existing Buildings Committee, the East Bay Chapter of the International Code Council and other.

Condition of Existing Wood Materials

- Damage commonly known as “dry rot” can occur to wood framing exposed to dampness or to water leakage. Termite infestation is another cause of damage to wood members. All buildings being strengthened where damage is suspected should have a thorough inspection but only those elements affected by retrofit need to be checked.
- If not repaired, “dry rot” and pest damage can weaken sill plates, studs, and wood siding and have a substantial adverse effect on a building’s response to earthquakes.

- Even dwellings without dry rot or termite damage may have weaknesses due to poor construction quality. For example, insufficient nailing of plywood, OSB or diagonal sheathing will result in a structure that is unable to resist the forces imposed during earthquakes.

- Simply repairing the weaknesses will not be adequate if the condition of the existing wood framing members to be utilized is in doubt. Consequently, it is recommended that the exposed wood be thoroughly inspected to ensure that which is “part of the strengthening work” is in good condition. Members that contain splits, checks (cracks) or knots affecting the ability of the member to resist earthquake forces must be strengthened or replaced.

- Existing wood members showing evidence of fungus infection, commonly referred to as dry rot, or evidence of insect infestation must be removed and replaced. Fungus remains active even after the affected area is treated.

- Fungus infection can be found by probing the wood members with a sharp object like a knife or awl. If the probe easily penetrates the wood, the member might have fungus infection. Sound wood will be difficult to probe. In some cases the fungus infection will be found only inside the member because rot may affect the wood from the inside and progress outward. By the time it is noticeable on the surface as staining or softening, the wood’s strength may already be significantly degraded.

- Thus it is important to perform probing and visual observations prior to and during construction. In these cases, probing with a sharp tool will not always locate the infection (see Figure 2.).

- This concealed condition may be encountered when drilling for new sill anchors. If the drill suddenly moves through the wood, a pocket of fungus infection is likely to have been encountered. The portion of the sill plate containing the infection will need to be cut out and replaced with a new piece of sill plate.
- The new sill plate must be anchored in accordance with Tables 3-A and 3-B. When replacing pieces of sill plate, pressure treated lumber will need to be used to protect the new member from fungus infection.

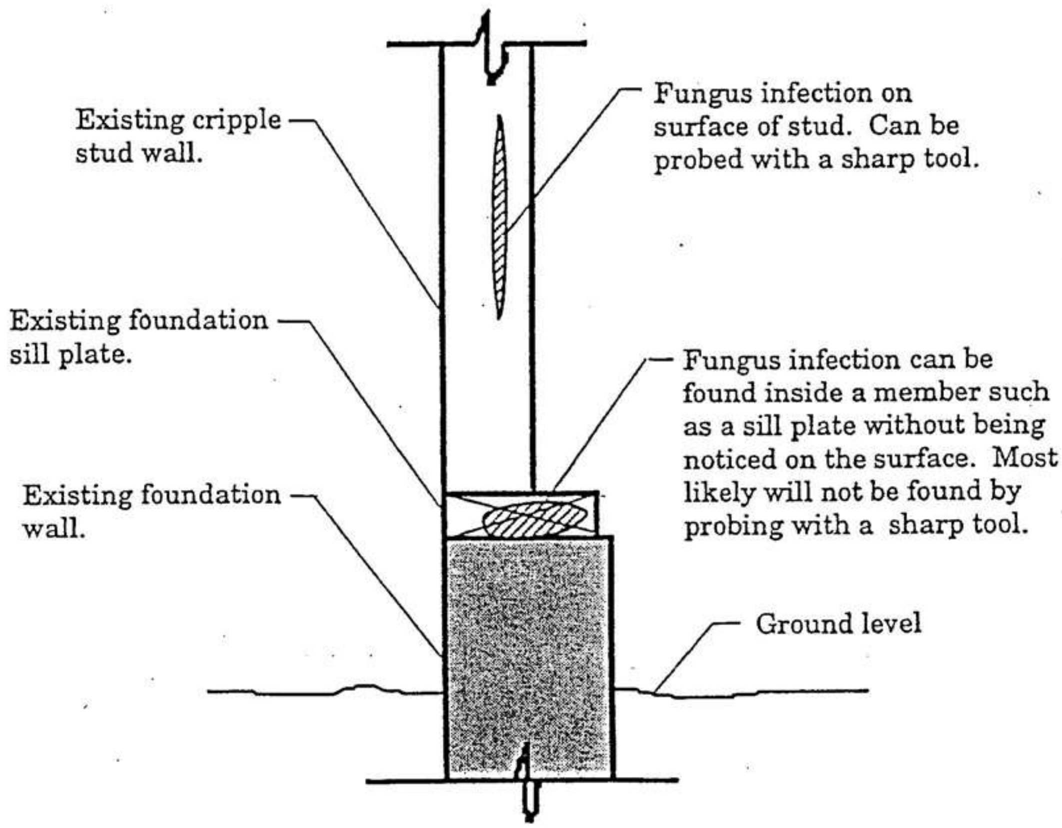


FIGURE NO. 2 - LOCATING FUNGUS INFECTION

- Fungus growth occurs where wood is made continually or repeatedly damp, by a leaking plumbing pipe, or by repeated saturation and drying from an exterior wall that leaks during rains. Simply removing and replacing infected wood will not necessarily prevent the fungus infection from recurring.

- It is important to find the cause of the leak, repair it, and allow the remaining wood to dry. Repairs will involve tracing a water stain or the actual water to a leaky pipe or fitting or other source.

- It is usually more difficult to trace a leak in the exterior wall covering. Hand held moisture detectors could be used to locate moisture intrusion. Spraying the exterior of the residence with a high-pressure hose and then using the moisture detector on the inside surface of exterior walls can locate defective flashing or torn paper backing behind the stucco.

- A common area to check for leaks through the wall is at building corners. Evidence of the infection may be found in other members that are not involved in the strengthening work. It is recommended that other members should also be replaced and the source of the wetness eliminated by appropriate repairs. Insect infestation, on the other hand, stops damaging the wood once the infestation has been stopped. Consequently, a member that has been significantly damaged by insects does not need to be removed if it can be strengthened.

- The easiest method of strengthening is to add a new member next to the damaged member.

- Unfortunately, there are no clear guidelines to indicate when insect damage requires strengthening. This determination must be based on judgment gained through experience.

- Prior to removing sill plates or studs for repair due to fungal or infestation mage, temporary shoring must be installed.

- The design of this shoring must be carefully planned by a qualified Design Professional with shoring design experience and installed by a contractor with shoring experience.

Floor Joists Not Parallel To Foundations –

- For these strengthening procedures to be effective there must be a continuous horizontal load path from the exterior walls to the foundation.
- Where floor joists are perpendicular to a cripple wall, or frame into the cripple wall at an angle, existing rim joists (or blocking) need to be connected to either the foundation sill plate (if there are no cripple stud walls) or the top plate of the cripple wall.

- When reviewing existing construction, if there is a connection between the rim joist and the plate that meets the nailing requirements of this section, such a connection may be considered adequate for this link in the load path.

- Where these connections do not exist, new connections must be made. Rim joists will need to be toe-nailed with 8 penny 2-1/2" long common nails, spaced 6 inches apart, through the joist into the plate. Blocking will need to be toe nailed. Use of proprietary products for these connections might be easier than toe nailing.

- When approved by the building official, these connections may be made by using products with current Evaluation Reports by an independent testing authority. Because the forces in a single-story structure are relatively small, it is not necessary to verify these connections if the blocking or rim joists are present.

- In multi-story buildings, the connections between the foundation and the blocking or rim joists must be verified. When these requirements are not met or cannot be verified, the provisions of this Chapter apply. In some cases, existing construction may not include a rim joist or blocking.

- In other cases the members are smaller in width than a nominal 2 inches (1-1/2 inches). In these cases, either a new nominal 2-inch wide full-depth joist or blocking, or one of the methods described in the Chapter may be used to provide the load path from the floor to the sill plate or cripple wall.

- In addition to providing a load path link, the rim joist or blocking provides rotational restraint for the ends of the floor joists.

Floor Joists Parallel To Foundations –

- Where floor joists are parallel to a cripple wall, the same load path concept applies as with joists perpendicular to foundations. In this condition, the end floor joist must occur over the foundation wall or cripple wall and be connected.
- If this member is not connected to the plate, it will need to be toe nailed with 8 penny common nails spaces at 6" apart or with equivalent approved hardware. This connection need only be verified for multi-story building, for which seismic forces are larger.
- If an end joist in a multi-story building is not connected to the sill plate on top of the foundation, or this connection cannot be determined, the end joist may be connected to the sill plate with sheet metal angles (proprietary hardware is available). Where clearances do not permit installation of this angle, an alternate method using _-inch plywood attached to the foundation plate or cripple wall top plate and to the underside of the flooring. may be used.
- Recommend deleting most of this paragraph, as it merely restates the provisions.

❖ **Foundations**

○ **New perimeter Foundations**

➢ **Foundation Evaluation By An Engineer or Architect**

- It might not be economical to replace existing partial perimeter foundations or unreinforced masonry foundations. In order to determine if existing foundation systems are adequate, an engineer or an architect would evaluate both the condition of the system as well as its ability to resist the prescribed forces. This analysis would be limited to the foundation system only.

- If other strengthening is to be performed, it must comply with the prescriptive provisions of this chapter.
-

Details for New Perimeter Foundations

- The first three weaknesses listed in Section 303 involve buildings without complete foundation systems. These conditions will be resolved by installing a new concrete or masonry foundation system around the perimeter of the dwelling. It is the intent of these provisions that all new foundations meet the current minimum standards of the Building Code.

- Although the Building Code sometimes allows plain concrete foundations for one- and two-family dwellings, standard construction practice is to provide nominal horizontal reinforcing. Reinforcing is often required where the soil is expansive. These provisions, therefore, require reinforcing of all new concrete foundations with a minimum of one No. 4 reinforcing bar in the top and bottom.

- The Building Code has specific provisions for minimum clearance under the structure for both access and ventilation. Occasionally, older construction did not provide the clearances that are required today. It is not the intent of these provisions to require current code clearance when a new foundation must be installed.

- To require excavation or raising the building would be extremely difficult and costly and will not improve its resistance to earthquakes.

- If substantial fungal or insect infestation has occurred in the past, the owner may want to consider measures to prevent future damage. In some cases, remedial work will be required per Section 304.1.2. An existing partial concrete foundation (weakness type 3 in Section 303) may be replaced or it may be evaluated by a Design Professional to determine if it can perform in a manner equivalent to a continuous foundation.

- An existing unreinforced masonry or stone foundation (weakness type 4) may be replaced with a new foundation that complies with the Building Code or it may be evaluated per Section 304.2.2. Replacement might be uneconomical or aesthetically displeasing. A well maintained unreinforced masonry foundation might be adequate to support a building for normal vertical loads, but its strength and ability to brace the building during earthquakes should be evaluated.

- If the existing unreinforced masonry foundation is not used to resist earthquake forces, a new foundation bracing system may be provided that is independent of the existing foundation. Examples include foundation capping and providing concrete plugs (alternate segments) in the existing foundation.

- Both fixes require partial removal of the existing foundation as well as shoring and/or jacking of the wood superstructure. The new system must be designed to resist all the earthquake forces from the building occurring at the foundation level. In this case, unreinforced masonry foundations do not require analysis and strengthening when an alternate foundation system is used.

- Concrete foundations are typically not reinforced and commonly have cracks due to shrinkage or long-term differential settlement. Even new footings have shrinkage cracks. Common locations for these cracks are at corners and near changes in footing height or thickness. Typical shrinkage cracks in footings are straight and vertical and have uniform narrow width. Isolated cracks less than 1/8-inch in width can be assumed not to significantly diminish the strength of the foundation. (ATC, 2002)

Foundation Sill Plate Anchorage

Existing Perimeter Foundations

- The provisions for connecting existing sill plates to existing foundations maintain the traditional code requirements for 1/2-inch diameter bolts spaced a maximum of 6 feet apart for one story buildings. Two and three story buildings need progressively more bolts because their height and added weight result in larger forces to be resisted.

- Expansion bolts and chemical anchors are acceptable for connecting to existing concrete. These connectors, due to their shorter length of embedment into the concrete, have lower capacity in concrete than anchor bolts that are castin- place when the foundation is poured.

- However, even with the required 4-inch embedment, these connectors will have the same capacity in the wood sill plate, which is the weakest link in this connection. Consequently, properly installed expansion bolts or chemical anchors can provide the same resistance against sliding as cast-in-place bolts.
- An expansion bolt is effective when the hole is drilled the correct size, the hole is relatively clean, and the bolt is properly tightened to set the expanding portion of the assembly in accordance with manufacturer’s specifications.
- For expansion bolts to be fully effective the foundation material must be able to engage the expansion portion without cracking. Where cracking indicates conditions of poor quality concrete or masonry ground during installation, expansion anchors may not be used.
- If cracks are observed during installation, installation should be 12 stopped, and a bolt should be installed at a new location at least 1 foot away. If the problem continues, chemical anchors or screw type should be used instead. All anchors must be installed away from the edge of the sill plate in order to be effective.

- The chemical anchor is a threaded rod that uses "epoxy" type adhesive to set the anchor. Chemical anchors are effective when the hole is the correct size and the hole is completely clean. Concrete dust must be removed in accordance with manufacturer’s specifications.

- A clean hole is more critical for chemical anchors than for expansion bolts. Chemical anchors are allowed for all types of foundations but are required where existing concrete is in poor condition or when the installation of expansion bolts causes cracking of the concrete.

- Some adhesives are viscous enough for use in horizontal holes but others are too thin and tend to drain out of the holes before setting. To avoid this problem, consult with current ICC Evaluation Services reports on anchor systems as well as manufacturers' instructions before purchasing.

- The provisions require square plate washers between the nuts and the sill plate. Because the holes that must be drilled to insert the connectors are larger than the connector diameter, the resulting hole in the sill plate is too large to provide proper bearing against the bolt. Consequently.

- The plate washer is installed so that the nut can be tightened sufficiently to develop the required clamping action between the sill plate and the top of the foundation wall. The use of plate washers is also intended to minimize the potential for crushing and splitting of the sill plate as the bolt is tightened.

- Due to the oversized hole in the wood sill a standard round washer does not engage enough of the wood around the hole.

- The use of the larger plate washer will eliminate the problem of the nut recessing into the drilled hole as it is tightened. Table 3-A provides the size of the plate washers required.

- The provisions call for the nut to be tightened to a "snug-tight condition" *after* epoxy curing is complete or *after* the nut has been tightened to set an expansion bolt. Tightening the nut to set the expansion bolt and tightening the nut to connect the sill plate to the foundation are separate operations.

- The setting requirements of expansion bolts vary according to the bolt used. The specific bolt manufacturer's procedures must be closely followed to assure that the bolt is properly set and is capable of transmitting forces into the foundation.

- Because these procedures vary, the provisions only address how tight the nut should be after an expansion bolt has been properly set or the adhesive of a chemical anchor has set.

- If the nuts are not tight against the washer plates, there will not be sufficient clamping action between the sill plate and the foundation wall.

- The nut should be tightened to the point at which the full surface of the plate washer is contacting the wood member and slightly indents the wood surface.

- Over-tightening beyond this "snug-tight" condition will cause crushing of the wood sill that will reduce the capacity of the connection. This section also gives the Building Official the authority to spot test the nut tightness during the required inspection.

- Also shows the condition of a battered footing. This type of slanted face footing will require that the wood shim installed between the steel plate and the wood sill plate must be shaped so the steel plate will have full contact against the shim when the lag screws are tightened. Further, a beveled washer under the head of the lag screw is needed to ensure that it bears fully on the steel plate.

- It is recommended that the shim be nailed to the sill plate (in addition to the lag screws), but the nailing must not split the shim. Pre-drilling of holes may be necessary.

- Alternative details may be easier and faster to install and should be acceptable in principle to the Building Official. Discuss potential alternatives with the building department or consider hiring a licensed design professional to prepare an alternative for unique conditions.

Placement of Chemical Anchors and Expansion Bolts –

- Careful attention needs to be given to the proper location and spacing of sill bolts. In order to assure that the sills are properly connected, this Section not only specifies the minimum spacing, but also limits the placement of bolts at the ends of pieces of sill plate. These provisions differ from those in the Building Code in requiring the bolts be placed no closer than 9 inches from the end of the sill plate.

- When bolts are placed closer than 9 inches to the end of a plate, there is a potential for that bolt to split the sill from the bolt hole to the end of the plate as the bolt is loaded from earthquake forces. When the bolt is placed more than 12 inches from the end of the piece there is a tendency for the end of the plate to lift due to overturning forces on the wall (CUREE, 2002).

- Placing the bolt between 9 and 12 inches from the end will minimize both tendencies. These provisions also address the realities that existing sill plates may be installed in short pieces either where the foundation wall steps or where new pieces of a sill plate must be installed to replace sections damaged by fungus infection or insect infestation.

- Therefore, the provisions specify a minimum number of bolts for various lengths of sill plate. It will not always be possible to install sill bolts at the exact spacing. There are many existing elements that can interfere with their placement, such as a fireplace, plumbing or mechanical ducts.

- The provisions of this chapter have taken these field situations into account and allowed that where physical obstructions exist, the bolts may be omitted. However, the spacing of the remaining bolts needs to be adjusted so that the same total number of bolts is installed as though the obstruction did not exist. It is recommended that if possible, the bolts with close spacing should coincide with the sheathing locations.

Cripple Wall Bracing

General –

- When bracing a cripple wall, consideration must be given to providing adequate resistance to both the horizontal forces and the tendency for uplifting one of the ends of the wall. Any wall panel that is subject to earthquake forces has a tendency to want to lift up at one end as well as slide.
- This uplift can be resisted by one of two methods. In new construction a "hold-down" anchor consisting of a heavy gauge metal angle is bolted to a stud and also anchored into the concrete foundation. Because this would be impractical to install in existing construction the method used in this chapter is based on the proper proportioning of the length and height of the cripple wall bracing panels.

- By making the panels longer, more weight from he walls and floor above can be engaged to resist the uplift force. The basic proportion required is a minimum length of braced cripple wall paneling at least two times its height. (RRR, 1992) In addition, longer panels are needed as the number of stories above the wall increases.

- This is simply because a taller building imposes larger horizontal forces on the braced cripple wall panel. Nonbearing walls, where floor joists run parallel to the wall, will not engage substantial weight, so hold downs at the ends of the walls may be prudent for multi-story buildings. Where cripple wall bracing panels can be connected at corners of buildings and installed in combined panel lengths longer than the minima defined in, the potential for wall overturning can be reduced. In addition, continuity provided by rim joists, plates, and floor framing tends to create appreciable fixity at the tops of the cripple walls that offsets wall overturning.

- To stay within the limits of these prescriptive methods, a maximum of 4 feet for the height of the cripple wall was established to limit overturning effects. When the height of the cripple wall exceeds 4 feet, the dwelling owner will need to have the bracing designed by a Design Professional.

Sheathing Installation Requirements –

- plywood is prescribed as the required sheathing because of observations of ruptured 3/8" thick (3 ply) plywood panels documented in the MMI VIII and IX intensity areas caused by the Northridge Earthquake (LA, 1994).
- Let-in braces have been observed to perform poorly in past earthquakes without some other form of bracing (LA, 1992). Let-in braces are no longer accepted by the 2003 IBC for Seismic Design Category D, E, and F. Therefore, walls braced only with let-in braces are considered a structural weakness requiring supplemental bracing. Even though the provisions accept existing diagonal wood sheathing to be acceptable (see Section C303), diagonal wood sheathing is no longer cost-effective for strengthening weak cripple walls.

- The omission of this material was not based on its ability to resist lateral forces, as it has performed well in past earthquakes and high winds (LA, 1992).

- Instead it was based on cost considerations and practicality, since this type of sheathing is more time consuming to install and more expensive than wood structural panels. Proprietary bracing methods may also be used when approved by the building official.
- The most important component of wood structural sheathing is proper nailing. To prevent splitting of existing wood framing 8d nails are considered optimum for 2x material. If splitting of studs is observed, periling of holes is recommended.

- Predrilled holes should have a diameter of about 3/4 of the diameter of the nail. Nail guns tend to produce less splitting than hand nailing.
- Minimum edge distance for nails should be maintained for plywood and the wood studs and top and sill plates to prevent splitting or premature nail failure. With this size nail, 4 inch spacing provides adequate capacity with the minimum bracing length permitted by Table 3-A and Figure 3-10. Further, using larger nails or closer spacing would, by comparison of capacity, require larger diameter sill bolts or closer spacing of the bolts than specified in Section 304.3.2.
- When plywood is installed on the inside face of cripple walls with an exterior surface of stucco, care must be used to prevent damage to the stucco. In this situation, it is recommended that 3-inch long #6 wood screws may be used instead of nails. The 3-inch length is needed to ensure that the shank (unthreaded) portion of the screw will have at least 5/8" penetration into the studs and plates. If the threaded portion of the screws exists at the plywood-stud interface, the screws can fail in a brittle manner when the earthquake occurs.

- If a nail gun is used, the operator must make sure that the nail heads do not fracture the surface of the plywood. Local variations in the density of the backing (new or existing wood framing members) can create situations where it will be difficult to maintain consistent nail penetration.

- The use of a flush head attachment on a nailing gun will usually prevent overdriving. When a nail head fractures the plywood surface, the amount of force that this particular connection is capable of resisting is reduced significantly. It becomes much easier for the nail head to pull through the sheathing material. Whenever a nail head fractures the surface of the sheathing, the nail must be discounted.

- When a nail is discounted, it must be left in place. Removing the nail will further damage the sheathing material and could result in the rejection of the whole sheet of sheathing by the inspector. (Shepherd, 1991) When purchasing structural sheathing, one of the structural grades must be stamped on the sheets used.

- The correct grade of structural sheathing is important. Refer to the Building Code for more information.

Distribution and Amount of Bracing –

- Bracing panels are required at or near each end of each wall line . Recommend using Fig C3-2 (see notes below) and definition. Although not required, it is beneficial if the cripple wall bracing panels align with the panels above the first floor to provide a more direct load path. Thus, cripple wall bracing panels should be located under windows only when necessary. Performance can also be enhanced if all panels along a single wall line are of similar lengths rather than having one very long panel and other shorter panels. This prevents concentration of forces in one location.
- The existence of a small number of studs over 14 inches in heights should not trigger this requirement. The requirement that buildings with cripple wall studs over 14 inches in height be treated as having an additional story helps reduce the overturning forces by requiring additional panel lengths. The 14 inches is intended to be an average.

Stud Space Ventilation –

- The most common form of cripple wall bracing will be to add sheathing to the interior face of the cripple wall from within the crawl space. When this is done, a closed space is created between each stud that does not allow natural ventilation.

- This can result in a buildup of moisture that will lead to fungus infection. In order to protect these concealed spaces from fungus infection, 2” to 3” diameter (3” recommended) ventilation holes must be provided.
- These ventilation holes will allow the free movement of air within the stud space thereby minimizing the risk of fungus infection.

- When 2x horizontal blocking is needed in the stud space to provide backing for panel joint nailing, it must be installed with the wide face oriented vertically, flush with the face of the stud on which the sheathing is being installed.

- This can be easily accomplished using commercially available fence rail hardware at each end to attach the block to the studs. This will eliminate blockage of ventilation inside the stud space. Ventilation holes should be cut or drilled as close to round as possible.

- Hole cutting tools are available to cut the required size hole. Square holes, or other shapes with sharp corners or notches can result in high concentrations of stress when the panel is loaded.

Existing Under floor Ventilation –

- Air circulation under the floor protects the framing from fungus infection. Vents by themselves do not provide all the solutions to under floor ventilation.
- It is imperative that there be cross ventilation. In order to have cross ventilation, the vents must be located in opposite walls, approximately opposite each other.
- In many cases, heating units have been added to the dwelling and the ducts are installed within the crawl space. When this is done, the ducts block the cross ventilation and significantly reduce the efficiency of the vents.
- Consequently, a dwelling with vents that meet existing code might not be adequate if there are obstructions to the cross ventilation. If this condition exists, consideration should be given to providing additional vents in order to obtain the necessary cross ventilation.

Quality Control –

- Strengthening work is only as good as the quality of the construction. In most jurisdictions the strengthening work required by this chapter will require building permits and inspections. Prior to requesting a permit the owner or contractor should survey and determine all existing conditions, dimensions, and other considerations significant to the retrofit or repair work.
- A plan should be prepared (11x17 inch paper with 1/8” = 1 foot scale is adequate) showing the location of proposed sheathing and spacing of anchor bolts.
- The drawings should differentiate between new and existing components. Because of the nature of the work being performed and the materials being used, there are some additional inspections that need to be performed that are not specified in the Building Code for new construction.

Placement and Installation of New Chemical Anchors or Expansion Bolts.

- The Building Official must approve the use of expansion bolts or chemical anchors. Building officials often use Evaluation Reports from the International Code Council Evaluation Services (ICC-ES) as guidance in the products they approve.

- These reports set allowable design values based on testing and specify requirements for construction quality control. They often call for special inspection, especially for bolts that might be subject to tension forces. (Special inspection generally involves inspection of the work while it is being performed, as opposed to when it is complete.

- It is performed by qualified individuals retained by the owner.) For the purposes of GSREB Chapter 5, special inspection is not required because the bolts in question are intended 16 to act primarily in shear, not in tension. Even if these bolts are not set exactly as noted in the evaluation report they will still work to resist the shear forces from earthquakes.

- The waiving of special inspection thus represents a justifiable cost savings. While special inspection is not typically required, the building official may still require verification of proper installation per Section 304.3.1. In lieu of special inspection, it is recommended that a post-installation torque test for expansion anchors be done together with inspection for bolt spacing, end distance and a spot check to make sure the nuts are properly tightened.

- Usually this inspection would be performed after the bolts were installed and before the cripple wall sheathing is placed. However, the building official may elect to perform this inspection at the same time they inspect the installation of the cripple wall sheathing. Vent holes in each stud space should be located and sized to allow inspectors to reach in and torque the bolts. Both expansion bolts and chemical anchors must be approved, as stated in the Section 302 definitions.

- This means that they generally must have a valid evaluation report from the International Code Council Evaluation Service (ICC-ES) or an approved equivalent independent test report. Normally chemical anchors require continuous inspection during their installation as a part of their approval for use.

- Continuous inspection checks that the hole is the correct depth and is sufficiently clean prior to placing the epoxy material. Its purpose is to ensure that e Since sill bolts are not subject primarily to tension, however, the provisions make an exception to this rigorous special inspection requirement.

- However, it is recommended that a less expensive torque test of expansion anchors in lieu of tension tests is an appropriate substitute for special inspection. Torque tests must be performed on at least 25 percent of the total number of expansion bolts installed and must be done in the presence of a building inspector or a deputy inspector employed by a testing agency that is hired by the owner and approved by the authority having jurisdiction.

Installation and Nailing of New Cripple Wall Bracing.

- The final required inspection is to make sure that the connections of the wall bracing panels are installed correctly and completely. The bracing serves no purpose if the connections do not engage the proper framing members, or, in the case of wood sheathing, are overdriven.

Work Subject to Special Inspection –

- The building official may require special inspection where conditions on a particular job site make inspections difficult . Retrofit and strengthening work often involves unusual conditions or proprietary components that require additional levels of quality control. In order to address these problems, the building official is allowed to require that a special inspector verify work. Most dwellings have some features that will not conform to the conditions and strengthening provisions used in this Chapter.
- To avoid situations where those existing conditions either preclude the use of these prescriptive provisions or where other complications may occur that would make their application to a specific building difficult; the Building Official is encouraged to perform a pre-design inspection. The cost of this inspection, however, may be in addition to the normal permit fee . Such an inspection might not be necessary if the owner provides adequate drawings supplemented by photographs to permit adequate review by the building official.
- The purpose of a pre-design inspection is to notify the owner or contractor of problems that may need the services of Design Professionals. It is not intended to be a consulting service to the owner. Typically, this inspection should focus on the following issues: Areas where obstructions in the crawl space along exterior walls might prevent installation of adequate lengths of bracing;

❖ Areas that may be questionable with respect to insect or fungal damage to wood members to be used in the strengthening;

- Foundations that may be questionable or clearly too weak to be effectively used for anchoring sill plates;

- Tests of nut snug-tightness on sill bolts;
- adequate rim joist or blocking conditions along exterior wall lines;
- Other concerns that the owner or contractor believes will preclude the use of the prescriptive details or methods described.

Phasing of Strengthening Work –

- The phasing of the strengthening work can often be phased when approved by the building official. A new permit will be required for each phase before beginning the additional work. Phasing may benefit owners with limited budgets or scheduling conflicts with other planned alterations. The strengthening work in
- Phase requires that the work be performed on at least two parallel sides and never on one side alone. This is meant to prevent rotation of the foundation anchorage, in plan, from seismic horizontal forces.

Tuned-Mass Systems for the Seismic Retrofit of Buildings Peter Nawrotzki

- Passive seismic control strategies are based on the reduction of energy, which affects a structure in case of earthquake events.
- Some well known approaches make use of frictional, plastic or other energy dissipating behaviour of special devices. The following presentation reflects some special ideas for the increase damping in order to improve the seismic performance of buildings.
- For this purpose additional-mass systems are proposed and their performance is investigated theoretically as well as on the shaking table.

- Usually these systems are considered as not suitable for seismic applications, but this thesis is no more valid as a general rule, if certain design approaches are kept. Tuned-Mass Control Systems (TMCS) can be used to control the displacements, accelerations and internal stress variables of a structure in case of earthquakes.

- The safety against collapse and defined states of serviceability of the structures can be achieved.

- This system can also be used for the seismic retrofit of existing buildings as the inside of the structure is usually not objective to modification. Hence, the usual operation inside the building may go on during the upgrade activities.

- A well accepted strategy in utilizing seismic control systems is based on the increase of structural damping. As a first idea damping devices can be installed solely.

- Then, they have the task to damp the relative motion between two structures, two parts of the same structure, or the structure and the 'rigid' vicinity.

- The damping effects may be obtained by friction, plastic deformation or viscose behaviour inside the device. The entire improvement of the seismic performance becomes obvious by different national and international standards. Some well known curves are compared, provides an idea of possible control effects.

- Usually 5 % of critical damping can be assumed for buildings, and an increase of the damping ratio causes a reduction of the stress or acceleration response as indicated by the correction factor ⌊. As an example the increase from 5 to 20 % of critical damping would cause a reduction of the induced seismic responses by about 50 % according to the Japanese provisions.

- Tuned-Mass Damper Systems (TMD) are widely used for the reduction of vibration caused by wind and traffic like pedestrians or railway trains.

- Typical structures like slender bridges, stacks, high and slender buildings possess low levels of damping and may therefore undergo unacceptable vibration. TMDs cause control effects which are similar to the increase of damping. Depending on the mass ratio, the tuning frequency and the damping capability the amplitude reduction can be very significant and achieve values of about 10 to 20 % of the figures without TMD.

- The reduction effects in these applications are higher that in case of seismic events because the governing vibration is similar to stationary motions and the TMD gets better adjusted to the motion.

- Nevertheless significant reduction effects can also be observed for seismic excitation. The ideas of the improvement of seismic performance according to Fig. 1 can be confirmed by theoretical and practical investigations.

- In order to distinguish between ordinary Tuned-Mass Systems and those for seismic applications the expression Tuned- Mass Control Systems (TMCS) is used. The layout of such systems is slightly different from that for a usual TMD system. Here, the mass and tuning ratio as well as the damping is chosen according to different criteria.

- A typical situation for structures. Here, a multi-storey building is equipped with a tuned-mass system on the rooftop. The additional mass consists of reinforced concrete and

Numerical Investigations

- Numerical simulations of buildings under earthquake with tuned-mass systems have frequently been performed. In many cases a special building model is taken and the additional mass is connected with the building elastically; sometimes the mass ratio is varied. Then, different recorded earthquakes are run and the responses of the structure with and without tuned mass are compared. The obtained results are usually not showing a unique picture.
- It can be concluded from this procedure that the tuned mass improves the response behaviour for most of the investigated cases, but there are also models under seismic excitation without significant improvement.
- In all of the latter cases without significant difference the structural response without TMCS turned out not to be dangerous for the building. The reasons are the induced internal forces and acceleration responses which are at a low level without any need for further reduction. In these cases the governing natural frequencies are not excited. The described steps for the layout of a tuned-mass system do not reflect the required procedure for real projects!

- For real projects there is a building with columns, beams, frames, walls, floors, and other important members.

- The structure consists of certain materials, possesses certain dimensions and there is a certain mass or mass distribution, stiffness, ductility and many other mechanical parameters. On the other hand there is the seismic risk which can be described with statistical parameters.

- The most suitable representation for engineering purposes can be seen in a site specific response spectrum.

- Here, for instance, we can directly see whether the building is in the dangerous frequency range and furthermore we can derive artificial base-excitation functions which correspond to the project site.

- Also recorded seismic events can be taken for the layout of the tuned-mass of a real structure, but in these cases the acceleration-time histories have to be scaled according to the site specific response spectrum.

Concrete Building Structures

- The majority of buildings in regions of high seismicity in the United States do not meet current seismic code requirements, and many of these buildings are vulnerable to damage and collapse in an earthquake.

- Concerns for seismic rehabilitation of existing buildings grew considerably following the 1971 San Fernando earthquake and resulted in several programs to identify and mitigate seismic risks. and 1994 Northridge earthquakes provided significant new impetus for seismic rehabilitation of buildings in California and elsewhere in the.

- Earthquakes in other parts of the world provide a continual reminder of the need for seismic mitigation programs underpinned by research to demonstrate their effectiveness and improve the efficiency.

- Seismic rehabilitation research in the US includes individual investigator and coordinated program research efforts. The US National Science Foundation began to fund research on seismic rehabilitation in earnest in the early 1980s. The early efforts were not overtly coordinated, and it became apparent that these programs would be unlikely to comprehensively address the broad needs in terms of range of construction and performance objectives necessary for the development of research-based consensus design guidelines. In 1990, the National Science Foundation announced a five-year coordinated research program on seismic repair and rehabilitation of buildings.

- The objectives of the program were to provide information for evaluation of the vulnerability of existing structures for various levels of seismicity, and to develop economical construction techniques for repairing and strengthening hazardous structures.
- The program culminated with the publication of a special theme issue of Earthquake S.
- The NSF research effort was supplemented by research carried out at the National Center for Earthquake Engineering Research [e.g., Beres; 1996].

- In the 1990s the Federal Emergency Management Agency (FEMA) and the State of California separately began to develop seismic rehabilitation guidelines.

- These efforts were guided by research reported to date or under way at the time. For the FEMA effort, the American Society of Civil Engineers subcontracted a research synthesis project resulting in a compilation of previous research in electronic format.

- The California effort resulted in a research synthesis specific to concrete buildings [Mohole, 1994]. Applications of the FEMA 273 Guidelines [FEMA, 1997] and California-directed ATC 40 Guidelines [ATC, 1996] to rehabilitation projects has revealed additional research needs, several of which are being addressed by ongoing research [e.g., Meoble, 2000].

- The symbiosis between researcher and practitioner is leading to rapid advances in the state of the art in seismic rehabilitation the US.

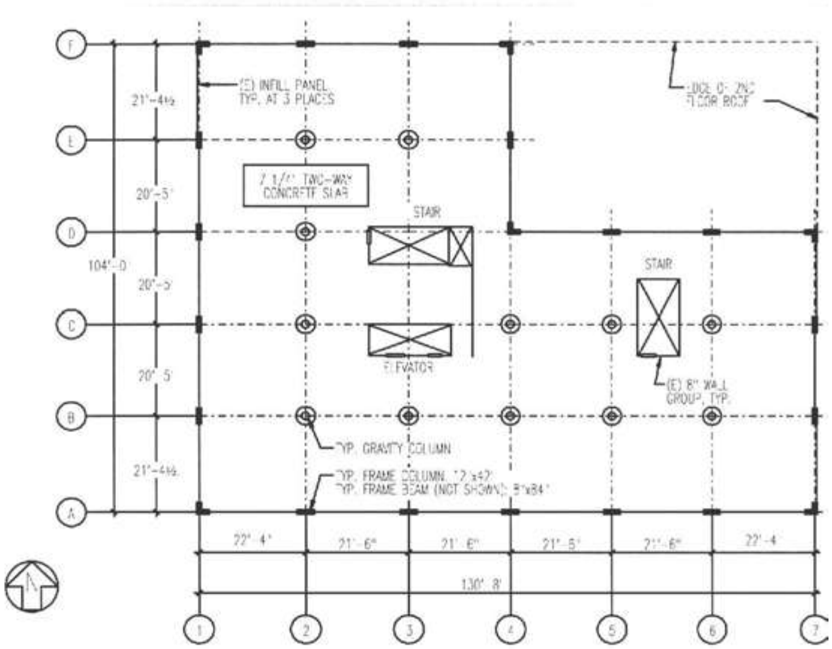
- The balance of this paper describes typical configurations of concern for existing concrete buildings, performance observations from past earthquakes, rehabilitation approaches, and rehabilitation research, with an emphasis on US conditions and research.

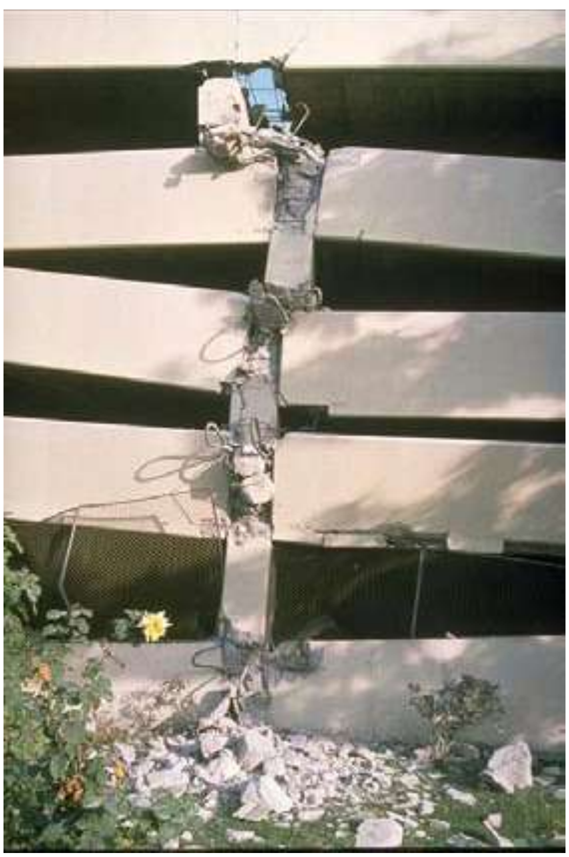
Typical Configurations and Details

- The development of details suitable for seismic resistance of concrete buildings was a gradual process in the US, and continues today. Main advances in understanding were made in the 1960s with the publication of the text by.
- While publication of this text along with 1960s and 1970s editions of the Structural Engineers Association of California (SEAOC) Blue Book resulted in some improvements to building design practices, it was not until the 1976 Uniform Building Code [UBC, 1976] that ductile detailing practices became mandated in the western.
- The specified details were surprisingly similar to those of today. Buildings constructed prior to that time commonly have significant deficiencies in configuration and detailing.

- Typical frame details in pre-1976 buildings in the western US are illustrated in Figure 1. Longitudinal reinforcement in beams commonly was discontinuous, and that in columns normally was lap-spliced with short length just above the floor level.
- Transverse reinforcement generally was not proportioned to prevent shear or lap failures, and details usually included wide spacing, open stirrups, and hoops with 90-degree bends. Joint transverse reinforcement was uncommon.
- All these details can lead to performance with inadequate adequate lateral displacement ductility as well as inadequate protection against vertical collapse.

- Most existing concrete buildings in the highly seismic western US comprise a mix of beam-column frames and shear walls; frame buildings are not typical.
- A floor plan of a representative building showing beam-column and shear wall framing. While the walls may provide most of the lateral resistance, not insignificant resistance may arise from the beam-column frame. Current practice usually aims to include the contribution of the beam-column frame so that rehabilitation is minimized.
- Whether this is the case or not, current practice requires that the beam-column frame be demonstrated to sustain gravity loads without collapse for design-level events.





Column failures – Northridge earthquake

Joint failures – Northridge earthquake

Slab-column connection failure – Northridge earthquake

- The most significant failures of reinforced concrete buildings in past earthquakes have been attributed to column failures. Causes have included column shear distress, spelling of column end regions, buckling of column longitudinal reinforcement, and formation of soft stories.
- Several collapse of one or more stories of buildings have been attributed to column failures (e.g.,

Failures of beam-column connections also have been observed.

Depicts an example from the Northridge earthquake Failure of slab-column connections have been observed in past earthquakes, in some cases leading to building collapse. The example shown in Figure 5 is of a waffle slab without continuous slab reinforcement through the column. Other examples of solid slabs, reinforced and prestressed, have been reported.

- Damage to shear walls and to coupling beams, while costly and disruptive, generally have not resulted in building collapse, and therefore have received less attention than have columns, joints, and slab-column connections. Failures in structures, while sometimes attributable to specific details, often have more systemic causes.

- Attachment of architectural elements, such as the parapet walls in the parking structure of Figure 3, can increase stiffness of components in specific locations of a building resulting in overload an premature failure. Weak-column/strong-beam systems are prone to story failures, especial frames having columns with widely-spaced Excessive flexibility in frames, as well as in frame-wall structures with flexible foundations may result in failure of framing components owing to excessive drift. The dividing line between damage without collapse and damage with collapse has not been identified analytically ties.

- The Great Hanshin-Awaji Disaster (Kobe Earthquake) caused huge damages to building structures, especially to old or non-engineered buildings. It has strongly been recognized that the strengthening of these seismic vulnerable buildings is one of urgent issues for the reduction of earthquake disaster.
- Thus, in response to the precious lessons from the Kobe Earthquake, the Japanese government enacted "the Law for Promotion of Seismic Retrofit of Buildings," in December 1995.
- In accordance with the law, existing buildings of more than certain floor area for public use shall be retrofitted to satisfy the seismic performance level equivalent to the current code requirement at the time of renovation.

- Practical evaluation of seismic performance and retrofit design of existing buildings has been based on “The Standard for Seismic Evaluation of Existing Buildings,” and “Guidelines for Seismic Retrofit Design,” before and after the enacting of the law, which are published from the Japan Building Disaster Prevention Association.
- Conventional methods for seismic retrofit of building structures are provided by the guidelines in detail, the effectiveness of which has been verified through past experimental research. On the other hand, various efficient methods of seismic retrofit have been developed or invented especially after the Kobe Earthquake.
- Although the effectiveness of the new methods was verified through various performance tests by the researchers in the developers group, neutral and standardized evaluation of the methods was necessary information to users such as structural designers or clients.

- For this purpose, the Japan Building Disaster Prevention Association (President: Tsuneo Okada) has set up a technical committee for evaluation of methods for building disaster prevention. The committee (Chairman: Shinseki Omani) officially started in 1996, and evaluated 28 methods in total until 2004.

- The members of the committee from 2004 are listed below. Most of the methods for evaluation are recently developed techniques for seismic retrofit or strengthening of old reinforced concrete buildings in Japan.

- Requirements and guidelines for design and construction by the new retrofit methods are prescribed as a manual in practice. The validity of the manual is evaluated, such as in the viewpoints of reliability of material properties, member performance, design equations, detailing and construction work.

- The scope of the method is also clearly restricted by reviewing background research. The details on the new methods could be available from the manuals in Japanese. However, the comprehensive information in English on the methods was not available.

- The tenth anniversary of the Kobe Earthquake. Commemorating the anniversary, various international events are planned, such as the International Symposium on Earthquake (ISEE Kobe 2005) and at United Nations World Conference on Disaster Reduction in Awaji and Kobe, in order to reduce seismic disaster in the future.
- The promotion of seismic retrofit of structures worldwide is one of the major topics to be discussed there. To contribute to the symposium of the conference above by introducing the seismic retrofit technologies recently developed in Japan, 22 methods out of 28, evaluated by the JBDPA committee, are outlined in English and compiled in this volume.

- This volume is prepared voluntarily by each developers group according to the given standard format for distribution at above meetings. Note that the views shown in this volume do not reflect those of the committee members but those of the developers. Also please note that some of these outlines in English introduce broader scope of application than approved by the JBDPA evaluation procedure. The contents is to be uploaded to the website of JBDPA soon and will be updated periodically in the future. The sincere cooperation and efforts of the developers for drafting the volume are gratefully acknowledged.

Technical Committee on Evaluation of Building Disaster Prevention Methods

(From April 2004 to present)
Shinseki Omani (Chiba University, Chairman)
Yoshihito Abe (Tohoku Institute of Technology)
Toshikatsu Ichinose (Nagoya Institute of Technology)
Daisuke Kato (Niigata University)
Toshimi Kabeyasawa (University of Tokyo)
Takashi Kaminosono (MLIT)
Kazuhiro Kitayama (Tokyo Metropolitan University)
Hiroshi Kuramoto (Toyohashi University of Technology)
Takeyoshi Karenina (Taisei Corporation)

Hiroyasu Sakata (Tokyo Institute of Technology)
Osamu John (Hokkaido University)
Shinichi Sugawara (Science University of Tokyo)
Norio Suzuki (Kajima Corporation)
Matsutaro Seki (Ohibayashi Corporation)
Hideo Tsukagoshi (Shimizu Corporation)
Shigemitsu Hatanaka (Mie University)
Shizuo Hayashi (Tokyo Institute of Techonology)
Masaki Maeda (Tohoku University)

Research for verification

The effectiveness of CRS-CL Method for strengthening reinforced concrete columns has been verified Were conducted for the total of 78 specimens in the first to tenth phase, which represents reinforced concrete or concrete-encased steel composite columns in old buildings of Japan or ridge columns of highway and railway.

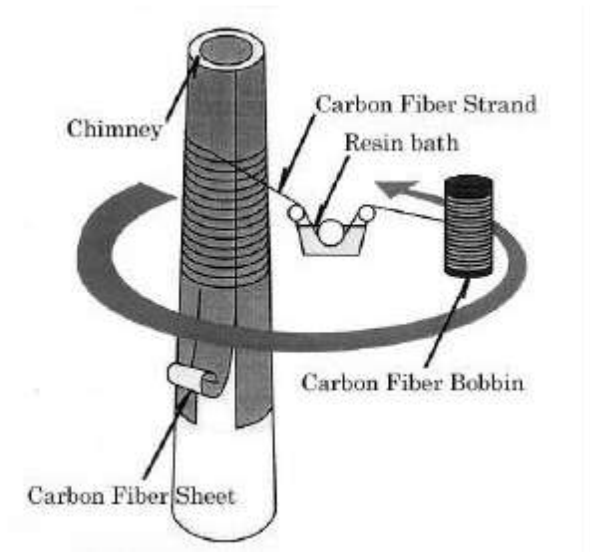
Many of the columns were strengthened using carbon iber sheet or strand. The columns strengthened by the new method could maintain lateral and axial load capacity until more than eight percent inter-story drift, while the bare specimens without strengthening failed in shear or bond at small drift and graduate lost axial load capacity. The typical hysteresis relations are compared for a bare reinforced concrete specimen and a CRS specimen as shown in Figure 2. The specimens after tests are shown in Photo 1.

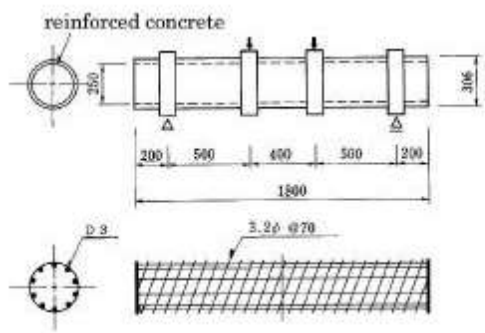
Concrete prisms nd cylinders confined with the CFRP material were also tested to grasp the confinement by FRP. Through these test series, the method is verified to be effective for prevention of the loss of lateral and axial load capacity under various structural conditio ns. And also, a structural design guideline for CRS-CL Method was established. A shaking table test was conducted for the verification of the new strengthening method under dynamic loading.

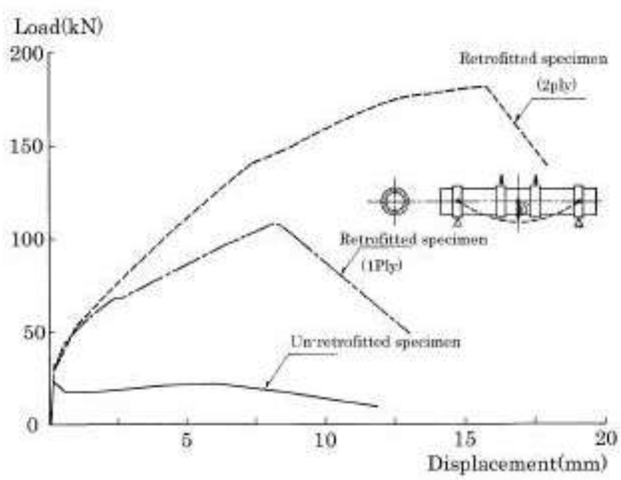
Each specimen consists of four reinforced concrete columns and a loading steel slab and tested on the shaking table at the Institute of Obayashi. One was a bare reinforced concrete column specimen designed to fail in a brittle manner, while the other was strengthened by the CRS-CL Method. The bare columns without strengthening failed in shear resulting in collapse associated with loss of the axial load capacity.

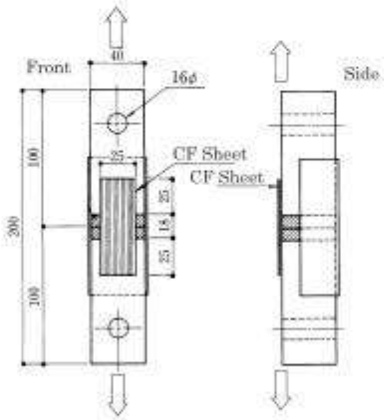
On the other hand, the columns strengthened by CRS Method survived against three times of the same input motion. The bare reinforced concrete columns and CRS-CL columns after the shaking table test are shown in Photo 2.

RC column (b) CRS-CL columns

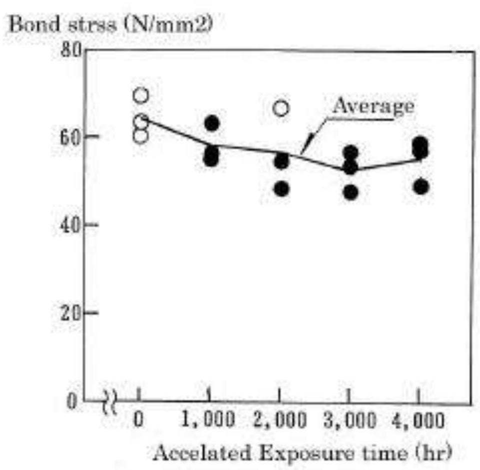

Chimney
Carbon Fiber Strand
Resin bath
Carbon Fiber Bobbin
Carbon Fiber Sheet


reinforced concrete
250
306
200
500
400
500
200
1800
3.2φ @70


Load(kN)
200
150
100
50
0
5
10
15
20
Displacement(mm)
Retrofitted specimen
(2ply)
Retrofitted specimen
(1Ply)
Un-retrofitted specimen



Specimen of exposure test relationonship



Research for verification

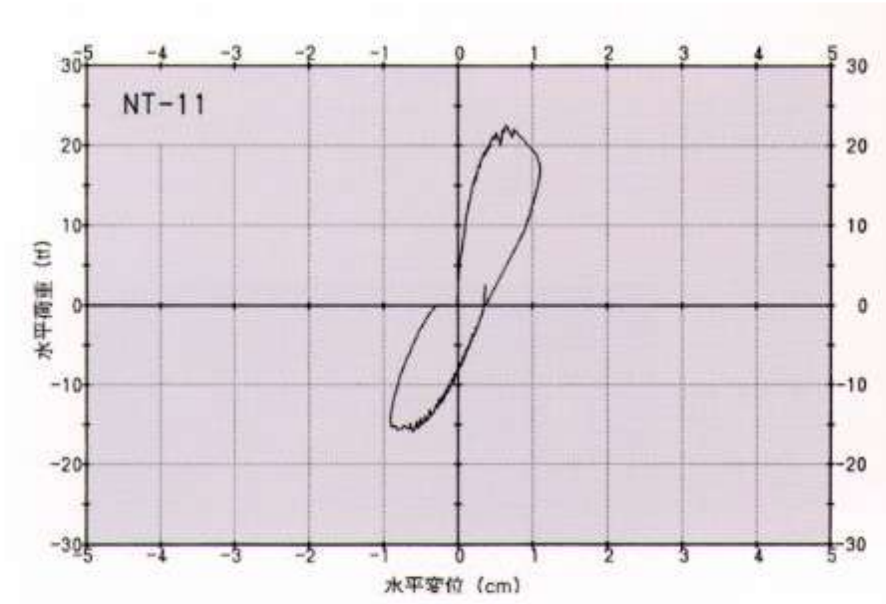
The effectiveness of MARS for strengthening reinforced concrete columns has been verified through a serious of seismic tests. Tests on columns were conducted for nineteen specimens, which represents reinforced concrete columns in old buildings of Japan or worldwide. Some of the columns were strengthened using carbon fiber sheet. The columns strengthened by the new method could maintain relatively high gravity load until more than ten percent inter-story drift, while the bare specimens without strengthening failed in shear at small drift simultaneously losing axial load capacity.

The typical hysteresis relations are compared for bare reinforced concrete specimen and MARS specimen as shown in Figure 2 with after tests photograph. Various types of concrete prisms and cubes confined with the sheet were also tested, based on which the resistance mechanisms of the columns were interpreted. Through these test series, the method has been improved to be effective to prevent the loss of capacity not only against axial load but also against lateral load reversals.

(a) RC (without carbon fiber sheet)
(b) SRF (with carbon fiber sheet)
Figure 2 the typical hysteresis relations and photograph
Of RC

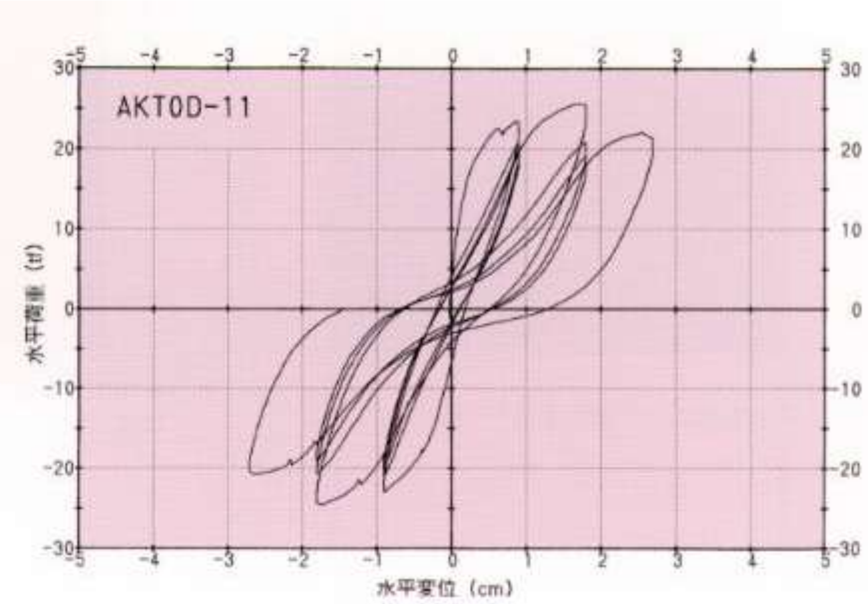
Research for verification

The effectiveness of AF System for strengthening reinforced concrete columns has been Verified through a serious of seismic tests. Static tests on columns were conducted for Many specimens varying amount of fibers to establish design methods. Figure 4 (a) Shows a bare RC column without agamid fiber sheet which represents columns in old Buildings. The columns strengthened by AF System (Figure 4 (b)) could carry higher Lateral load and show ductility with flexural failure while the bare RC columns failed in Shear with relatively low deformation. The typical hysteresis relations are compared for Bare reinforced concrete specimen and AF specimen as shown in Figure 4.



Lateral deformation (cm)

(a) RC (without agamid fiber sheet)



Lateral deformation (cm)

(b) AF (with agamid fiber sheet)

(c) The typical load-deformation relations of RC column and AF column

(d) Further research have been continued to develop applications of aramid fiber retrofitting
(e) method after the certification of AF System by JBDPA. Research themes about aramid
(f) fiber sheet developed by many organizations are listed as follows:
(g) Shortened columns by spandrel wall
(h) Columns subjected to high axial force (N/bd=0.6)
(i) Columns with finishing mortal
(j) Columns with wing wall
(k) Columns with low strength concrete of fc=6N/mm2
(l) Lateral deformation (cm)
(m) Load (tf)
(n) Lateral deformation (cm)
(o) Load
(p) Axial behavior of axially loaded columns
(q) T-shaped girders with slab
(r) Anchorage for agamid fiber sheet with steel plate and bolt
(s) Application of super high quantity sheets up to 1500g/m2
(t) Steel plate and agamid fiber sheet composite method
(u) For example, Figure 5 shows a shortened column by spandrel wall reinforced with
(v) Agamid fiber sheets after lateral loading tests. The specimen under constant axial load
(w) Maintains maximum lateral force up to deformation of 10% of column height.

Short column with agamid fiber sheet

Examples in practice

AF system is certified by JBDPA for existing building columns that require Strengthening for shear. But applications of agamid fiber sheets have been widely Expanding not only for building columns but also for bridge piers, tunnels, and Chimneys etc. More than 500 structures have been retrofitted with agamid.

Shear strengthening of building columns

Strengthening and repair for girders and slabs

Strengthening for bridge piers

Repair for chimney

Typical construction details
The construction process of CRS-BM Method is as follow: removal of coating, Preparation of surfaces with chamfering of corners, setting of bolts, application of Primer, wrapping CFRP, setting plates, and finishing with coatings. Figure 1 illustrates Wrapping CF sheets.

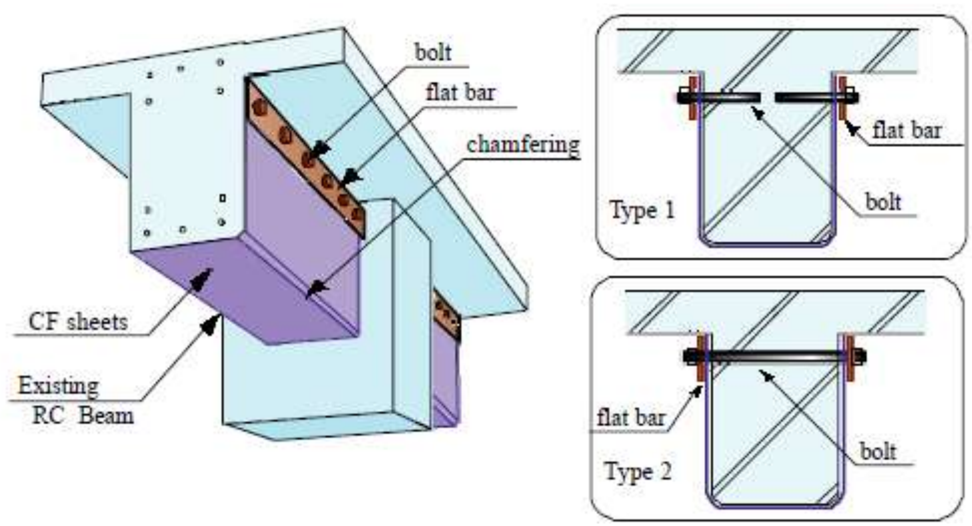
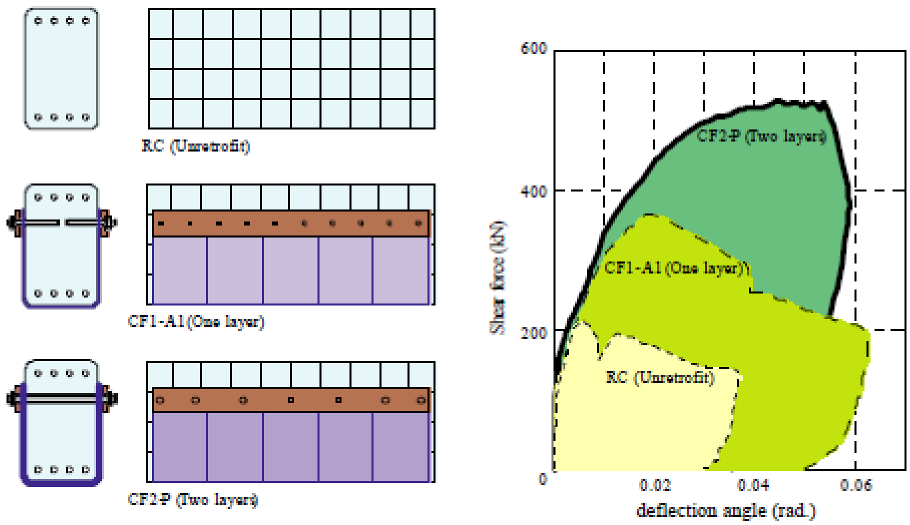


Figure 1 Typical construction details

Research for verification

The effectiveness of CRS-BM Method for strengthening reinforced concrete beams has Been verified through a series of seismic tests. Static loading tests on beams were Conducted for eight specimens in the first phase, and eight and five in the second and The third, which represent reinforced concrete beams in old buildings of Japan. Many of The beams were strengthened using CF sheet. The beams strengthened by the new Method could maintain lateral load capacity until more than 5% drift, while the bare Specimens without strengthening failed in shear at small drift. The typical hysteresis Relations are compared for a bare reinforced concrete specimen and a CRS-BM



(a) Anchoring methods of CF sheets **(b) Relationships between shear force and deflection angle**

Figure 2 The typical hysteresis relations of RC beam and CRS-BM beam

(a) unretrofit beam **(b) retrofit beam**

Photo 1 Specimens during and after tests

properties of the composites.

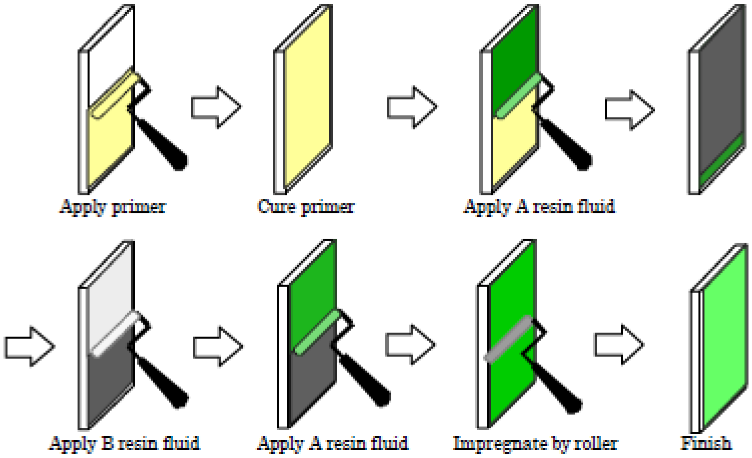


Figure 1 Concept of ABA-TCM

RC pillars wrapped with CF sheets were tested for shear. The test variables consisted of the number of CF sheet piles and two load conditions as summarized in Table 3. The test series specimens were applied a static axial load and a cyclic lateral load and symbolized. The details of the tests are illustrated in figure 2. CF sheets were wrapped around RC pillars in the hoop direction and the lap joint lengths were 200 mm.

The load-deformation curves of the test series specimens are shown in Fig 3. The maximum forces and the ultimate deformations on specimens wrapped with CF sheets

Typical construction details

Figure 1 shows a schematic diagram of the SR-CF system. The innovative technique called CF-anchor is used in this system. The CF-anchor is a bundle of carbon fiber strands which are strings of 2 to 3 mm in diameter consisting of 24,000 or 12,000 filaments. There are two types of CF-anchors. One is penetrating type, and another the fixing type.

The penetrating anchors are used for the shear strengthening of columns with wing walls. A bundle of carbon fiber strands is penetrated through a hole drilled at the wing wall. The ends of the bundle are spread like a fan and adhered to the carbon fiber sheet which is previously applied on the column. The bundle joins the both ends of the carbon fiber sheet, which was separated by the side wall. Consequently, it is made possible to envelop the column by the carbon fiber without demolishing a part of wing walls. Beams with slabs can be strengthened by the same method.

The fixing-type CF-anchors are used for the shear strengthening of walls. The carbon fiber sheet is adhered on the wall surface diagonally. An end of the CF-anchor is spread like a fan and adhered to the carbon fiber sheet. The other end is inserted into a hole drilled on the peripheral reinforced concrete frame and is fixed with injected epoxy resin. The CF-anchors fix the edges of the carbon fiber sheets on the wall to the peripheral columns and beams.

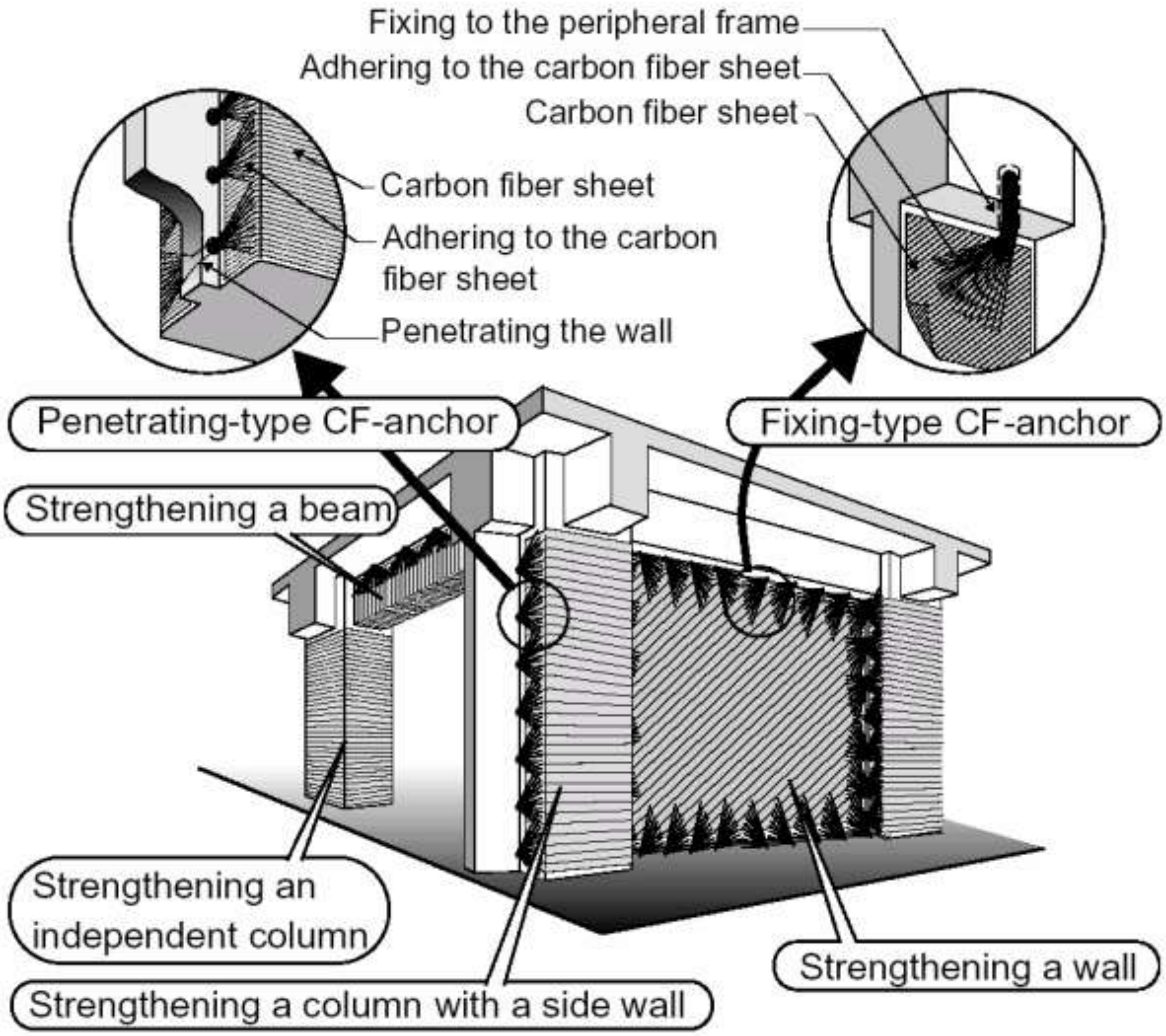


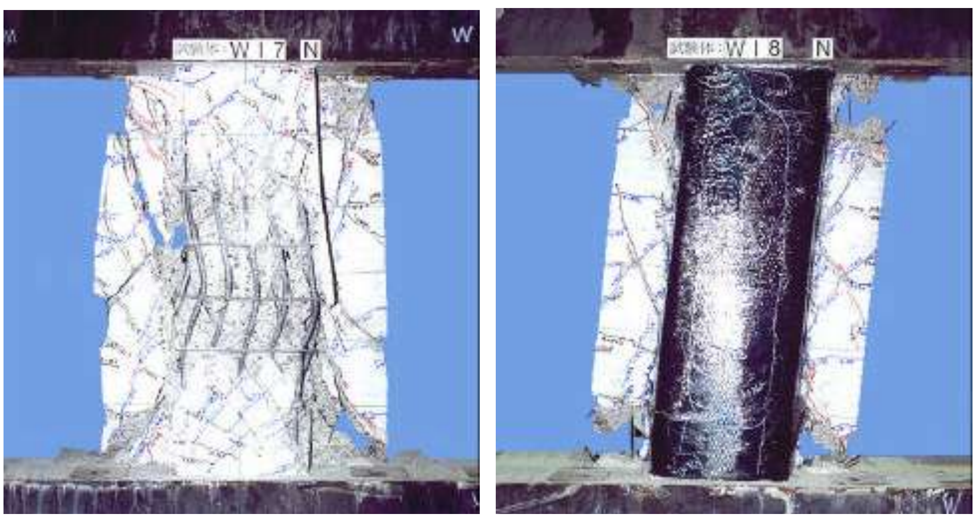
Figure 1 Schematic diagram of the SR-CF System

Research for verification

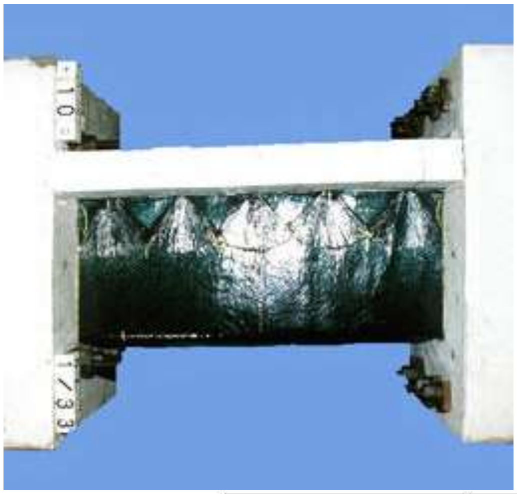
A number of specimens were tested to evaluate the effect of strengthening for each type Of structural members, such as independent columns, columns with side-walls, beams With slabs, and walls. Photo 1 shows specimens of columns with side-walls after the Tests. (a) is a specimen without strengthening and (b) is a specimen strengthened by the SR-CF system. The specimen without strengthening failed in sheer at small drift angle. On the other hand, the column of the strengthened specimen was not so much damaged even at 3.0% of drift angle while the wing walls were considerably damaged. Figure 2 shows the test results of the wall specimens. It shows that the shear strength of the strengthe ned walls increase in proportion to the amount of the carbon fiber sheets on the wall surfaces.

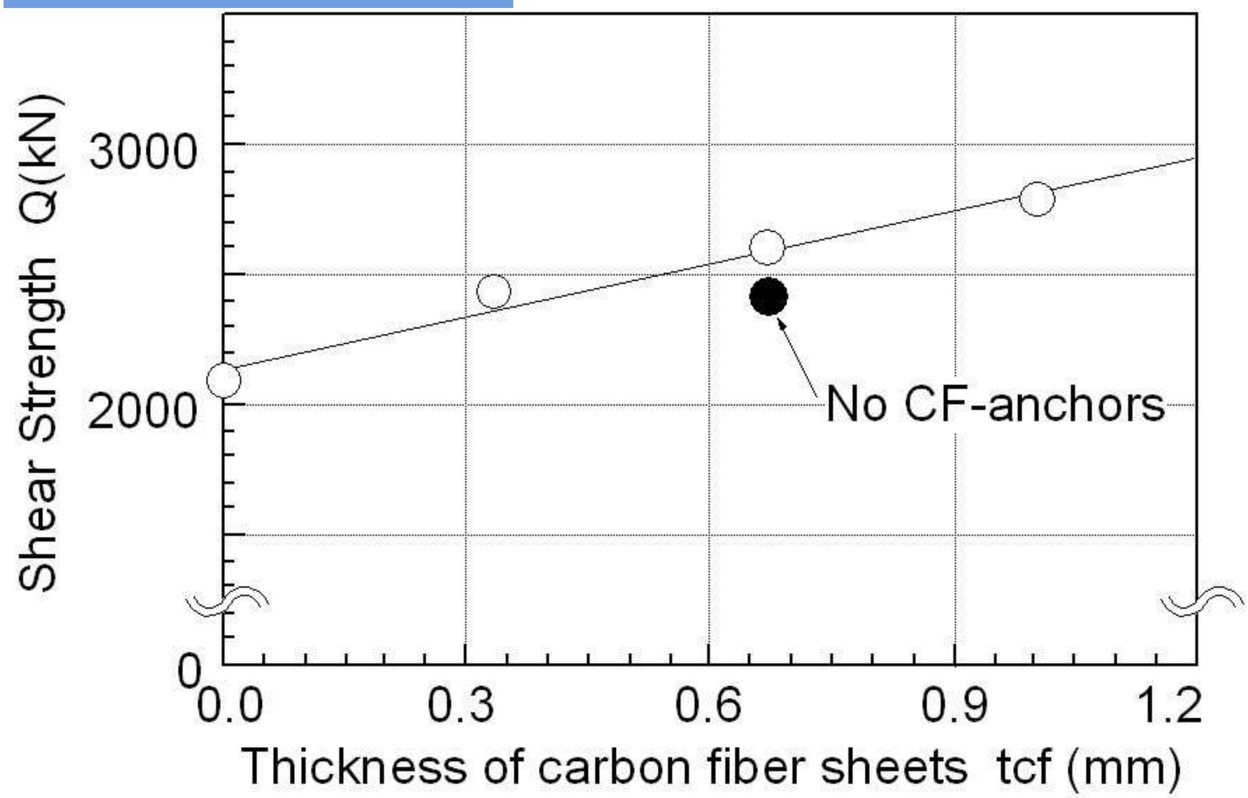
Examples of Application

show the execution of CF-anchors to the column with spandrel walls and windows. The CF-anchors are penetrated through the walls at the narrow space between the column and the window frame (a). As a result, the strengthening was completed without temporally removing the windows (b). Photo 4 shows a strengthening of a beam and Photo 5 shows a strengthening of a wall.



(a) Specimen not strengthened (b) Specimen strengthened by SR-CF system
Failures of column with wing-walls specimens after tests





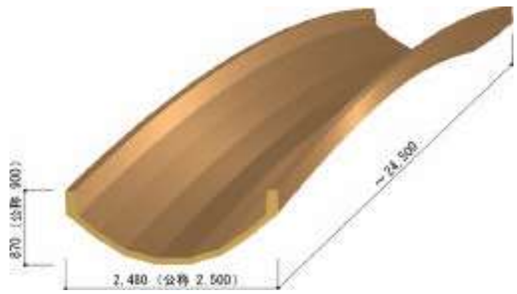
Beam specimens after tests
Test results of wall specimens

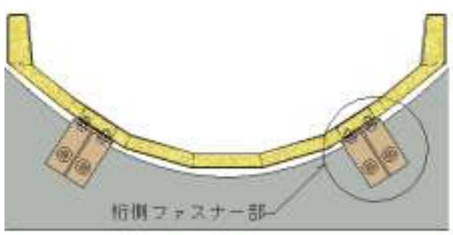
Allowable Unit Stress

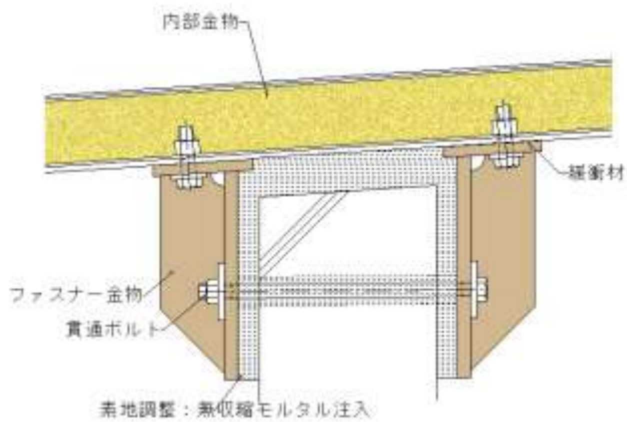
The sandwich structure can exhibit local buckling of the skin under bending or Compression. In case of an one-way hyperbolic paraboloidal shell, the design criterion is Usually due to compressive stress of the skin. Table 2 shows material strengths, standard strengths and allowable unit stresses. Material strengths are decided by coupon tests. Standard strengths (F) are decided by the product of the average of coupon tests and the statistical coefficient of 0.72. This coefficient is more wide range than three-sigma. The Design of CFRP sandwich panel roof is based on allowable stress design. Because of the brittleness of CFRP materials, the safety factors are decided as 4 for sustained loads and

Typical construction details

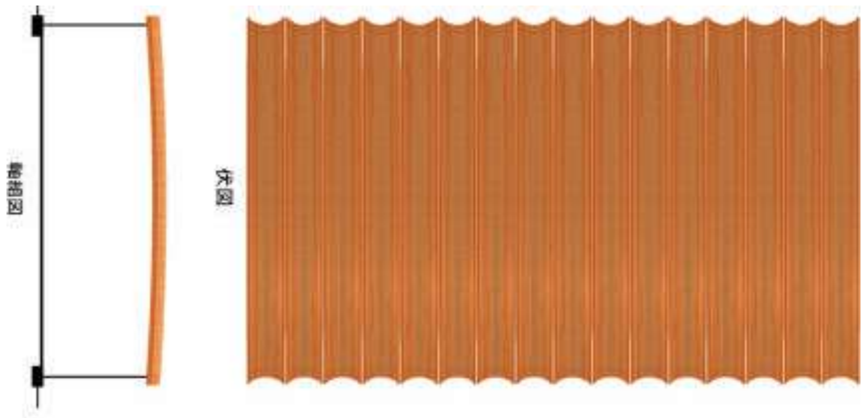
The specifications of CFRP sandwich panel roof is
The shape of the shell. The original shape is out of a hyperbolic paraboloidal surface.
Figure 3 and 4 show the connections of shell-tie beam and shell- shell respectably.
Figure 5 shows a roof plan and a section.







桁側接合部詳細図



Summary of the Method

The PITACOLUMN method is a completely external type earthquake-proof reinforcing Method targeted at low- to mid-rise reinforced concrete buildings. This method is a low cost, Short construction period and environmentally friendly method. Since it is not required to Works inside a building for reinforcement physically at each phase, no transportation of interior Equipment and removal and/or installment of existing fixtures are needed. Therefore it allows Continuous use of the building. Reinforcing structure is a reinforced concrete member containing a steel plate, and the method is surpassing in maintenance and does not require any Special finish comparable to that of the existing structure. The reinforcement working procedure Is as follows; drive post-installed anchors into the external surface of the existing structure, attach a steel plate by using these anchors and arrange reinforcing bars around the plate, and then cast concrete to complete.
One of two types of reinforcing pattern can be selected according to performance based Reinforcement volume or reinforcement target. That is a frame reinforcement is targeted at Ductility reinforcement and a frame reinforcement with braces is targeted at strength Reinforcement.

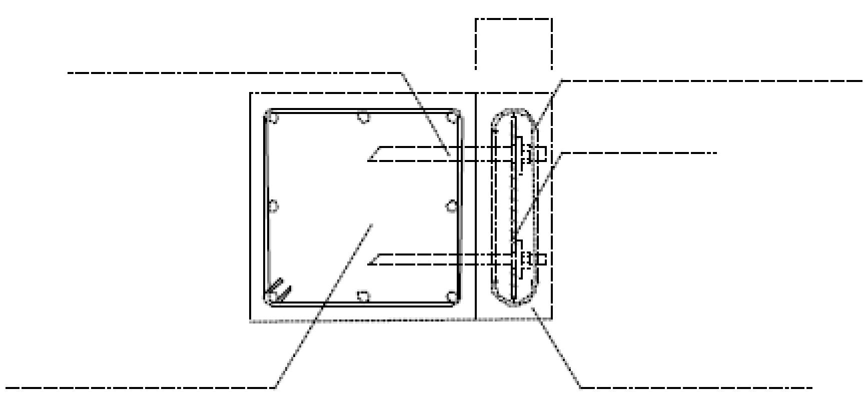


Figure. Summary of the Method

Specifications of Materials

The standards for the specifications of materials for the PITACOLUMN method are shown in Table 1.

Experimental Verification

The test specimens of 1/3 size model of 2 layers and 1 span were made and were tested with static loading to confirm the effectiveness of this method. The test results of the brace type reinforcement are shown here as the representative example.

The view of testing are shown in Photo 1 and the final state of cracking is shown in Fig. 3. The cracks in the existing parts were due to shear failure (accompanying bond splitting failure) of columns as with a test specimen without reinforcement, and it was observed that there is no change in the failure modes. And all cracks on the reinforced side are bending cracks only.

The relationship between load and displacement is shown in Fig. 4. The vertical axis is load and the horizontal axis is displacements of 2 layers. A dashed line in the figure indicates the test specimen without reinforcement. The test specimen with reinforcement was improved in both strength and ductility significantly compared to the test specimen without reinforcement and it was confirmed that it showed stable hysteresis up to drift angle of R = 1/60. Maximum strength (Max load) was 500 kN at drift angle R = 1/60. Though the brace of 2nd floor was buckled at R = -1/460 with Q =-183 kN during loading of R =-1/60, it sustained the increase of load and showed Q =-340 kN at R =-1/60. After that the brace of 1st floor was buckled at R = 1/44 with Q = 477 kN during loading of R = 1/30, and then its deformation progressed rapidly and led to a failure. It was also confirmed that the yielding process of members was preceded by the yielding at reinforced part and then the existing part was yielded.

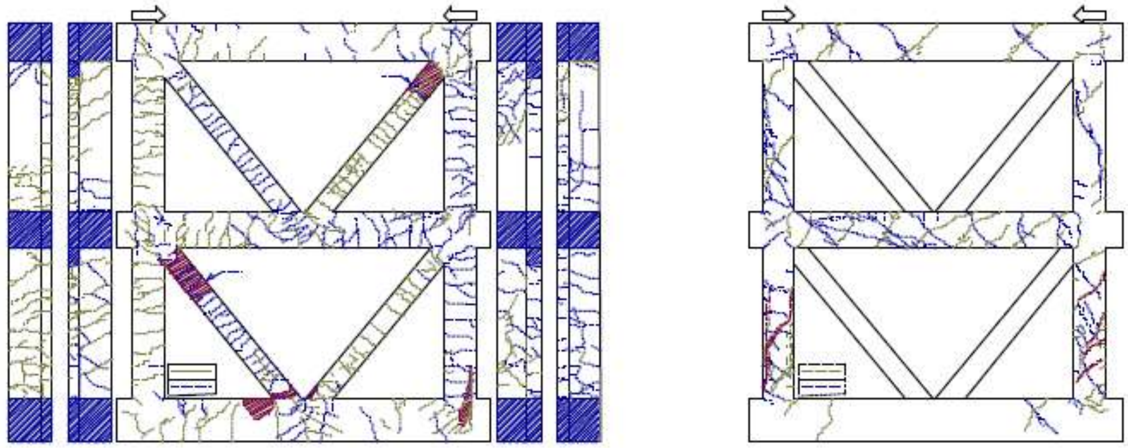


Figure.3 – Final State of Cracking

Construction Examples
There are 120 construction examples reinforced by this method, mainly applied for school buildings, to date

a)Junior High School Building (School Colored) b) Junior High School Building (Designed by Students)

c)University Building (with Exterior Panels) d)Elementary School Building (with Balconies)

Photo 2 – Examples of Reinforcement

Outline

In this seismic retrofit method, external energy dissipation braces (braces consisting of steel pipes and friction dampers embedded in the pipes) are installed on the external walls of an existing building to be retrofitted so as to absorb earthquake energy input to the building and thereby enhance its seismic performance (Photo 1). One advantage of this method is that by using friction dampers as response control devices, earthquake energy can be absorbed efficiently even under small amplitudes of the story drift angle (around 1/2000 to 1/1500 rad). This makes it possible to reduce the maximum response story drift angle of existing buildings to about 1/200 rad. Since friction dampers absorb earthquake energy efficiently, damper strength; frictional force of the damper, can be specified to 200 to 400 kN per unit so that the expected effect of strengthening can be attained simply by installing external damping braces to the external walls of the building. Although the conventional strengthening methods require removing sashes and interior and exterior finishes and reinstalling them, the newly developed method makes it possible to use the building without interruption while retrofit work is in progress. Thus, manpower requirements can be reduced significantly, and both cost and construction period can be also reduced. In addition, this retrofit method is environmentally considerate because the volume of waste materials to be disposed at which the interior and exterior parts are removed is small and noise level is low.

1. Title

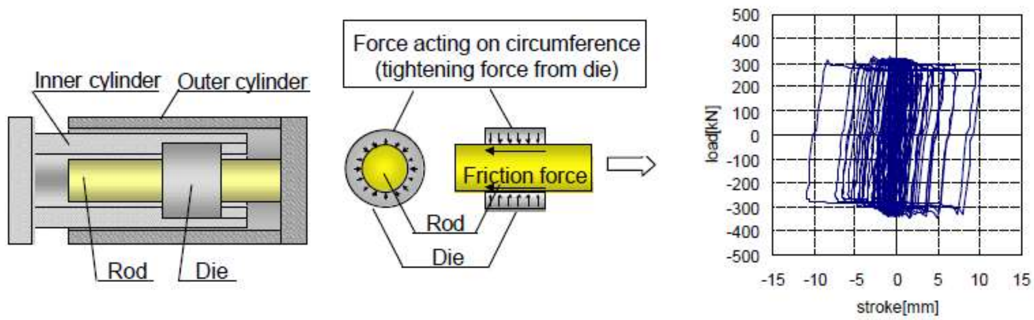
Aoki Seismic Retrofit Method by means of Energy Dissipation Braces

2. Outline

In this seismic retrofit method, external energy dissipation braces (braces consisting of steel pipes and friction dampers embedded in the pipes) are installed on the external walls of an existing building to be retrofitted so as to absorb earthquake energy input to the building and thereby enhance its seismic performance (Photo 1). One advantage of this method is that by using friction dampers as response control devices, earthquake energy can be absorbed efficiently even under small amplitudes of the story drift angle (around 1/2000 to 1/1500 rad). This makes it possible to reduce the maximum response story drift angle of existing buildings to about 1/200 rad. Since friction dampers absorb earthquake energy efficiently, damper strength; frictional force of the damper, can be specified to 200 to 400 kN per unit so that the expected effect of strengthening can be attained simply by installing external damping braces to the external walls of the building. Although the conventional strengthening methods require removing sashes and interior and exterior finishes and reinstalling them, the newly developed method makes it possible to use the building without interruption while retrofit work is in progress. Thus, manpower requirements can be reduced significantly, and both cost and construction period can be also reduced. In addition, this retrofit method is environmentally considerate because the volume of waste materials to be disposed at which the interior and exterior parts are removed is small and noise level is low.

3. Performance of friction dampers

Fig. 1 shows the configuration, mechanism and the hysteretic properties of the friction damper. The friction damper consists of a die, a rod, an outer cylinder and an inner cylinder. The diameter of the close- fitting rod in the die is slightly larger than the inside diameter of the die so that the rod is able to move in the axial direction while maintaining a constant frictional force. Earthquake energy causing dynamic motions of the building is transformed into frictional heat and is thus absorbed while the friction damper built into the damping brace is moving forth and back in the axial direction. As shown Fig. 1(b), the damper shows hysteresis loops of the perfectly elasto-plastic type. Frictional forces show a slight variation as damper strokes are repeated, and energy-absorbing performance of the damper is clearly observed.



(a) Configuration and mechanism **(b) Hysteresis loops of damper**

Fig. 1 Friction damper

Typical construction details

Photo 2 shows an example of the external damping brace installation. External damping braces are installed on the existing structural frame (main structure) through anchorage bases. The space between the anchoring base and the side of the existing structural frame is filled with grout and the anchorage base is fastened to the main structure by using prestressing steel bars.

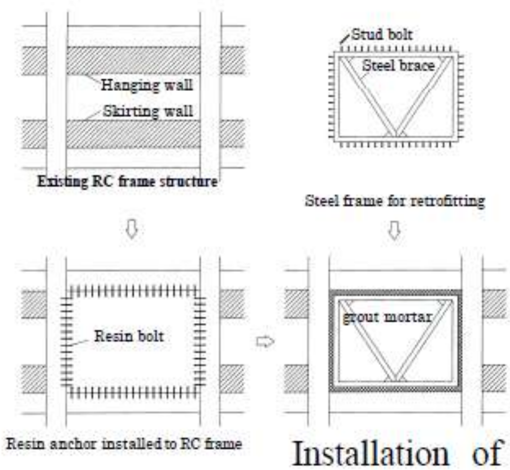
Installation method of external energy dissipation braces

Research for verification

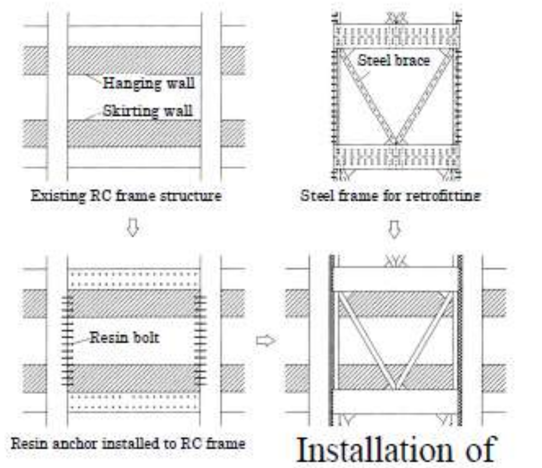
The effect of the damping retrofit method has been confirmed through the full-scale seismic tests using a three-story R/C school building planned to be demolished. The central part of the building in the longitudinal direction (three stories, 1 x 2 span) was used as a specimen, and concentrated loads were applied to the rooftop by the pseudo dynamic testing method. The effect of the damping retrofit on the building was confirmed by comparing the test results obtained without retrofit and the pseudo dynamic test results obtained by using the external damping braces with friction

Photo. 4 Application to an balconied school building

Photo. 5 Application to an apartment building



(a)Conventional retrofit method



(b)Yokosuka Type seismic retrofit method.

Specifications for materials

The present seismic retrofitting is consists of a steel frame structure with studs in which steel braces are connected, resin bolts inserted into the existing RC frame structure, and grout mortar which is filled between the steel frame and the existing RC frame.

H-shaped steel members are ordinarily used for the vertical members of the frame structure and the braces. Channel and angle shaped steels are used for the lateral members on the outer side of the floor and roof beams, respectively. Also, the studs are connected along the inside of the web portion of the steel frame structure for the purpose of enhancing the bond with the steel frame by means of grout mortar. Resin bolts are driven into the existing RC frame structure to which the steel frame structure with braces is connected so that they may lap with the studs connected along the web in steel frame. The grout mortar, which is filled between the existing RC frame and steel frame structures, possesses the property of non-shrinkage and high compressive strength above 30N/mm2 form the horizontal loading test a

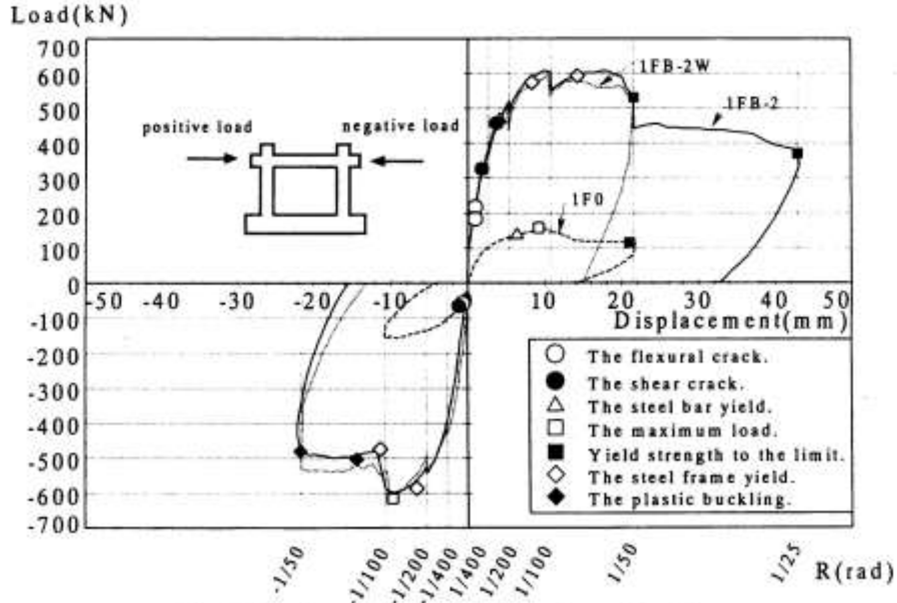


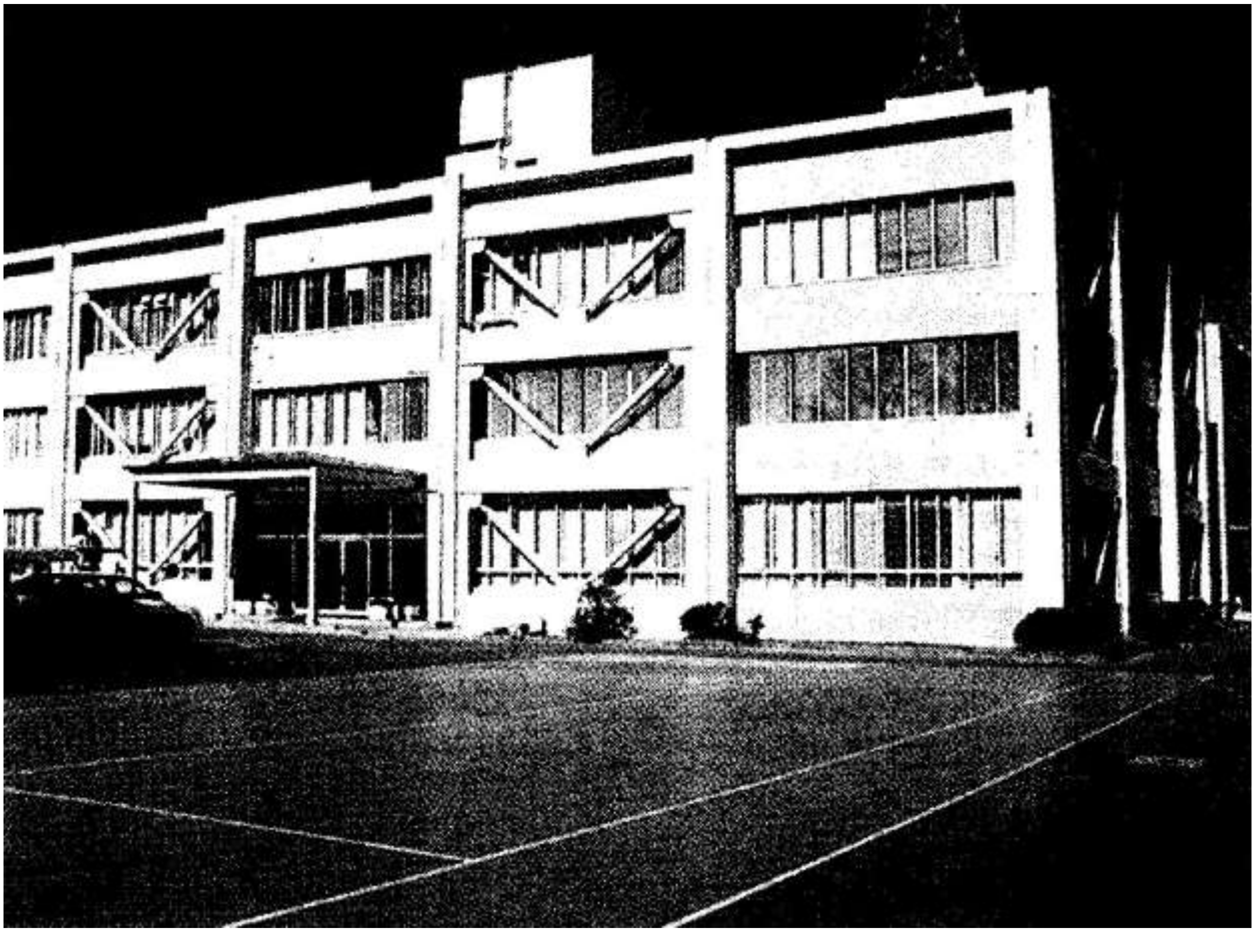
Fig.5. Envelope Curves for Mono-layer Specimen

two-layer specimen in Fig.6. From this, joint translation angle shows the maximum load at 1/100rad. Since the second layer column shows a resistance mechanism by the flexural failure, the deterioration of the strength after maximum load is not observed.

Examples in practice

The present seismic retrofitting method has been applied mainly to existing RC school and office buildings since 2001. Two examples of a four-story school and a three-story office buildings which were constructed in 1970 and 1967 respectively, are indicated by photographs 1 and 2. Damper devices are installed into the steel braces for seismic retrofitting in the latter building. The cost necessary for retrofitting by the present method is decreased about thirty percent in comparison with that of the conventional seismic method.

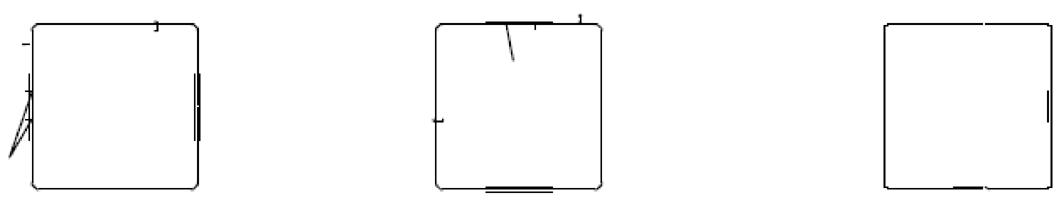
Retrofitted school building



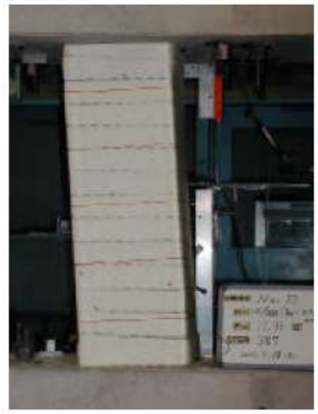
Retrofitted office building

Typical construction details

Fig. 1 shows the reinforcement details of the specimens of both series.



Reinforcement details



RC SRF

(a) Ultimate state (80% of maximum load)

RC (loss of longitudinal stiffness) SRF

(b) After test

Photo 1 Specimens during and after tests

(a) reinforced concrete column (b) SRF columns

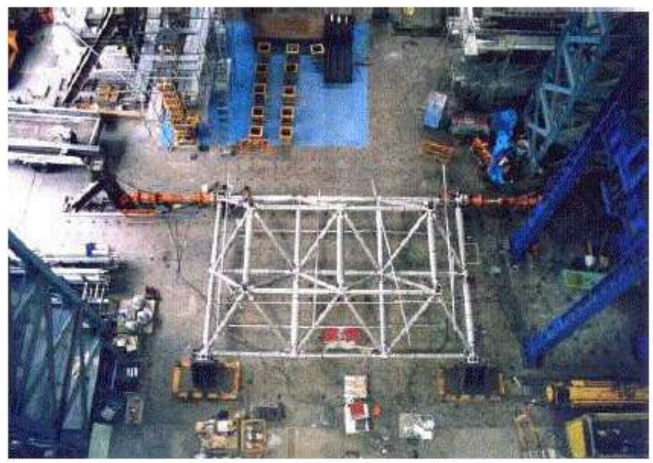
Photo 2 reinforced concrete column and SRF column after the shaking table test

Loading tests and investigations

Standard design of SNE-Truss was established based on the following loading tests and investigations.

Full scale loading tests of the simple beam of the space grid used in SNE-Truss was executed and a proportional and cyclic loading was applied to the unit space grids in order to investigate the buckling characteristics of the truss members with different member slenderness ratios. The loading test confirmed that the effective member slenderness ratio is predicted to correspond to 0.7 times the calculated ratio assuming the unsupported length L_K to be the distance between the grids. (Fig.5)

Regarding the double layer space grids latticed wall, to investigate the load carrying capacity, plastic deformation capacity and collapse mode, loading tests subjected to the in-plane direction were carried out. (Fig.6)



Simple beam loading test **In-plane loading test**

Specifications for the retrofitted frames

Two types of the infill are used, one is a steel framework with H section interconnecting braces and the other is a concrete shear wall. These infill are firmly connected to the inside of a bare frame by grouting with high strength mortar. Shear transfer at the horizontal connections simply depends on a resistance caused friction in compression area, because the basic measure of the TARS involves nothing but the grouting mortar to settle the infill. Some variations of constraints at the horizontal connections are described below if more enhancement of the shear capacity is required.

1. Constraining forces which are given with PC bars directly connecting the steel frameworks of upper and lower story.
2. Semi-cylindrical cotters which are carved top and bottom surface of a beam.
3. Embedded bolts which are fixed with the grouted mortar into the carved holes of a beam in the vertical direction.

Shear tests on the connection for a R/C beam and a steel of the framework were carried out to investigate its abilities of shear transfer with these constraints.

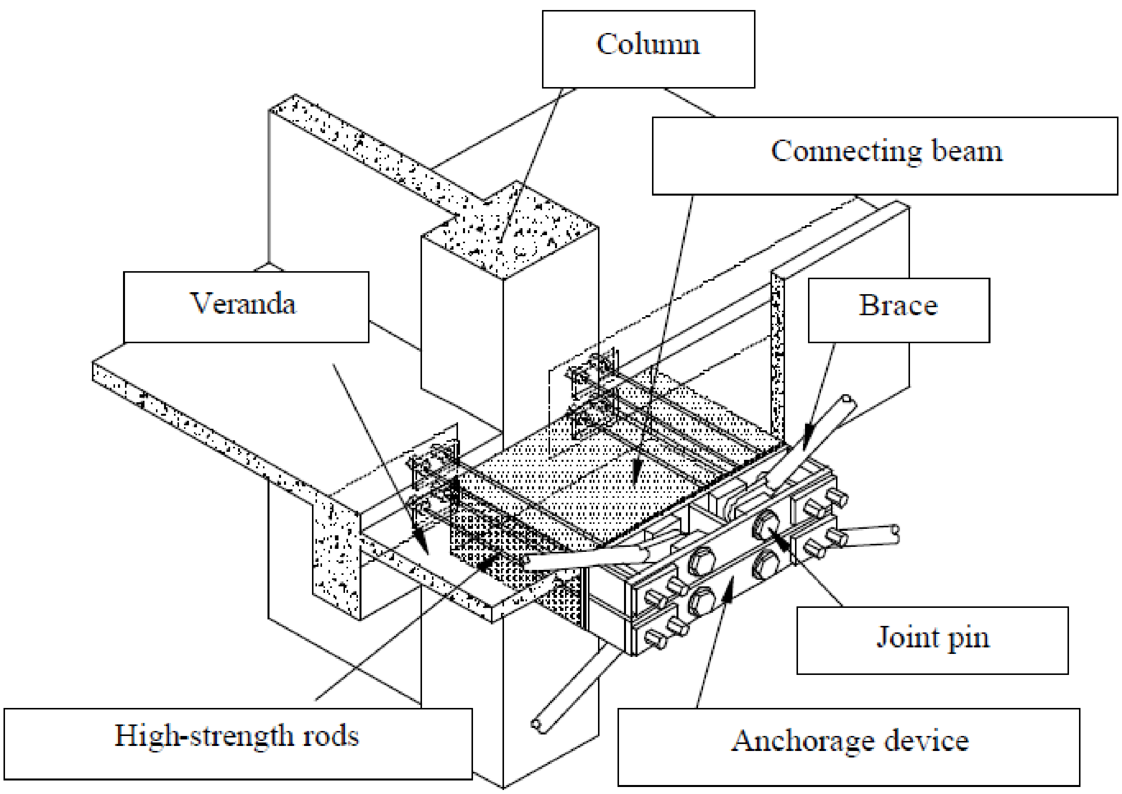
Typical construction details

The strong axis of the steel framework being arranged the in-plane direction of the beam,

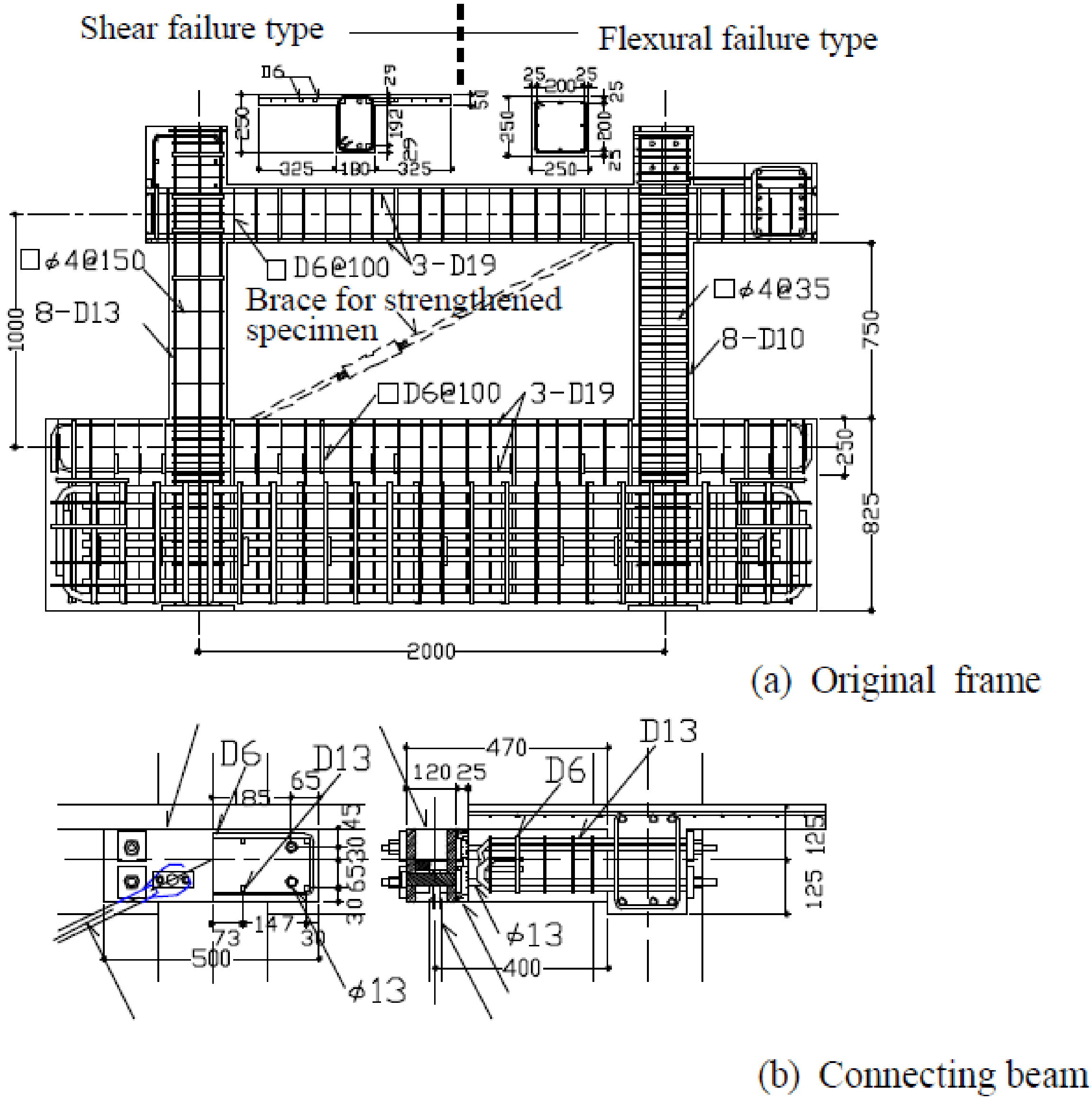
the stiffness of the framework is higher and the space grouted with mortar became narrower than those of a conventional measure.
The semi-cylindrical cotters, of which height is a ha lf of the radius, and the embedded bolts, of which embedded length into the beam is four times of the diameter, are placed at the horizontal connections to constrain a slip displacement.
Grouting is executed with premixed high strength mortar without shrinkage, which was designed to have a 7-day strength of 50 MPa.
The concrete shear wall is cast at sites but have no dowel bars at the wall-frame connections. The wall involves shear reinforcement of which ratio is around 0.3%, and the concrete which is specified to a strength of 30 MPa is used.
TARS facilitates to improve seismic performances in a shorter period, and easy construction procedure makes a reduction in a budget.

Typical construction details

The braces are attached outside the structural frames of existing buildings. The connecting beam was cast on the existing frame, and the anchorage device was jointed to the connecting beam. The braces were tied to the anchorage device with pin joints at both ends. The existing building, connecting beam and anchorage device were joined under normal stress conditions by pulling the prestressing bars.



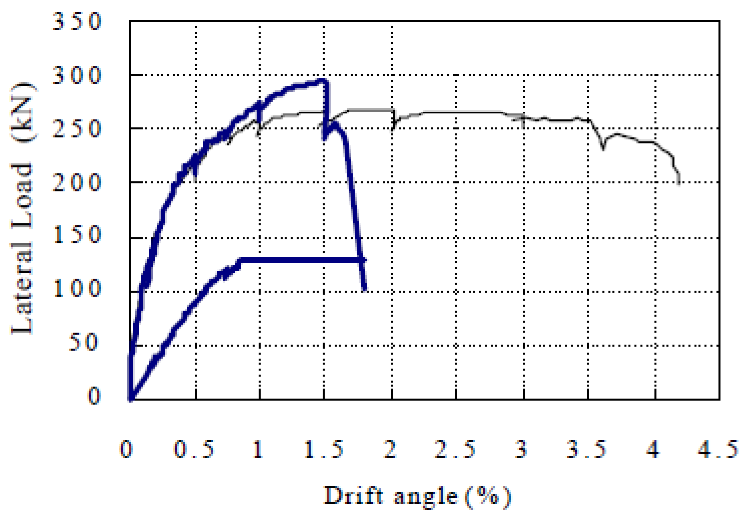
Outline of strengthening method



Details of experimental specimen (Unit : mm)

illustrates the lateral load-displacement behavior of each specimen. The horizontal displacement of the beam was divided by the height between the upper and lower beam centers to obtain the drift angle. The lateral load-displacement peak envelopes of each specimen are provided in Fig. 4. Herein, strengthened specimens No. 2 and No. 4 use the lateral load, subtracting the horizontal component of the axial load, which had been acting on the brace from the jack load measured with the load cells. The brace axial load was obtained by multiplying the section area, using Young's module, by the average strain of the brace. Original and strengthened specimens exhibited responses up to the lateral load-carrying capacity with very similar lateral load-displacement curves. This is seen in both shear and flexural failure type specimens. Lateral load-carrying capacity of the strengthened specimens can be estimated by adding the strength of the original frame to the brace strength

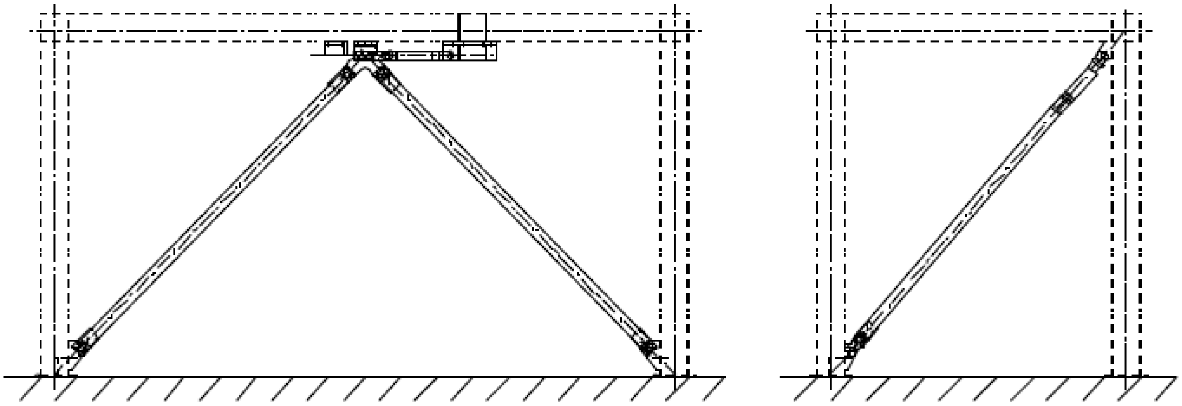
Lateral load-displacement response



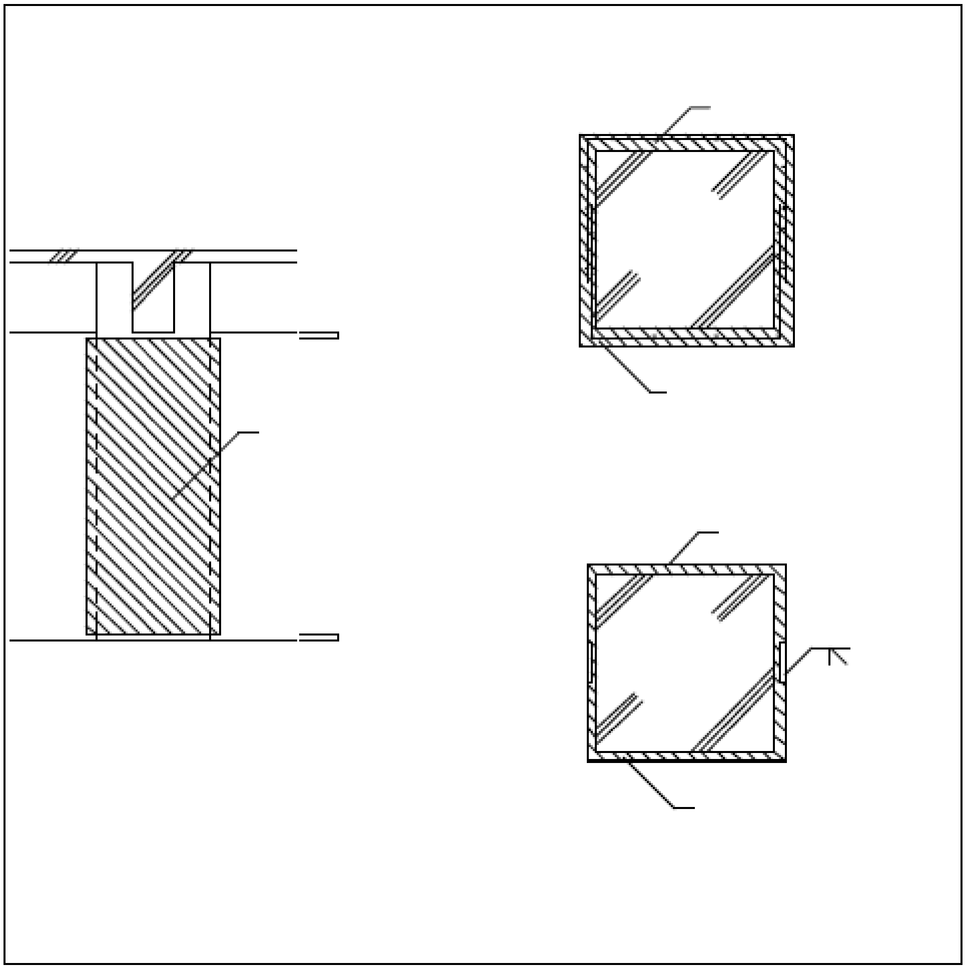
Shear force during an earthquake is transferred from the existing structure to the braces through connecting beams. Therefore, accurate evaluation of connecting beam performance is essential. Based on the biaxial loading tests for the connecting beam, a multi- linear skeleton curve can be identified and it is possible to evaluate the capacity of the beam and the drift angle at yielding of the brace.

Typical construction details

TFD is installed into a structure by two forms, shear- link and brace forms (see Figure 4). Ordinary details, not specially developed, used for steel structures are applied. However, when designing and constructing the details, it must be reminded that the advantage of TFD is shown only when the rigidity and strength of the connections between TFD and the existing structure are sufficiently secured.



Typical construction details



Column retrofitting with PPMG-CR method

A precast prestressed concrete (hereafter referred to as Pap) external frame seismic strengthening system is an a seismic strengthening method of building which heightens The shear strength of the building and raises earthquake resisting capacity by attaching The Pap frame to the existing reinforced concrete building (RC) or the existing steel Reinforced concrete building (SRC). An outline of this method is presented in Fig.1. The Pap frame is built up with the factory product precast concrete column and Beam by the post-tensioning system. Connection between the existing building and the PCaPC frame is carried out by floor slab or shear transfer block. The horizontal force And the axial force of the PCaPC frame are transmitted to the ground by the foundation Of the PCaPC frame, or by joining PCaPC foundation and the existing building one. This method is able to reduce the repair or restoration work inside the building on Account of connect the asismic strengthening member to the existing building from an Outside, and also produce well lighting and good facade of the building after reinforcement. Furthermore, using the precast concrete member lead to shorten the Construction term and improve in quality.