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Proceedings, Workshop on the Seismic Rehabilitation of Lightly Reinforced Concrete Frames Gaithersburg, MD June 12 - 13, 1995

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ABSTRACT

This report contains the proceedings from a workshop "Seismic Rehabilitation of Lightly Reinforced Concrete (LRC) Frames" sponsored by the National Institute of Standards and Technology and the U.S. Corps of Engineers, Construction Engineering Research Laboratory. The 1-1/2 day workshop was held on June 12-13, 1995 in Gaithersburg, Maryland. A total of 24 researchers, design engineers, and representatives from various federal agencies were invited to attend the workshop. The objectives of the workshop were to determine the state-of-the-art in the rehabilitation of LRC frames, to determine any gaps in the knowledge base that are preventing the development of guidelines or the widespread use of the rehabilitation methods, and the methods to fill these gaps. Six papers were presented at the workshop. The participants were divided into three working groups - concrete/masonry, steel, composites and damping systems. The participants discussed and recommended areas of needed research for rehabilitation methods in the three areas.

Keywords: Composite; concrete; damping system; frames; lightly reinforced; rehabilitation; retrofit; steel; workshop.

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EXECUTIVE SUMMARY

In the U.S., there exists a large population of structures that are classified as non-ductile, gravity or lightly reinforced concrete (LRC) frames. Common features of these frames include lack of continuity of the positive reinforcement in the joint, little or no transverse reinforcement within the beam-column joint resulting in inadequate confinement of the joint, low longitudinal reinforcement ratios for columns, poor positioning of column lap splices, low transverse reinforcement in columns, and overall frame proportions resulting in a strong beam-weak column collapse mechanism. These frames were not designed to resist significant lateral loads and are therefore vulnerable to lateral forces.

Research studies to correct the deficiencies of LRC frames have been conducted and a major guidelines development effort sponsored by the Federal Emergency Management Agency (FEMA) is currently underway. These guidelines are intended to provide rational engineering guidance on the necessary procedures when conducting a seismic rehabilitation analysis. However, there are still gaps in the knowledge base. To help determine and identify methods to fill these gaps, a 1-1/2 day workshop on the seismic rehabilitation of LRC frames was conducted on June 12 and 13, 1995 in Gaithersburg, Maryland.

The workshop was sponsored by the National Institute of Standards and Technology (NIST) and the U.S. Army Corps of Engineers, Construction Engineering Research Laboratories (CERL). The objectives of the workshop were to determine the state-of-the-art in the rehabilitation of LRC frames, to determine any gaps in the knowledge base that are preventing the development of guidelines or the widespread use of the rehabilitation methods, and to identify research needs to fill these gaps. The workshop participants consisted of U.S. and Canadian researchers and design engineers. Six papers were presented at the beginning of the workshop. The papers gave an overview of the state-of-the-art and required research needs and issues that needed to be addressed. The workshop participants were divided into three working groups: concrete/masonry, steel, and composites and damping systems. Base isolation systems were excluded from the discussions due to the limited time available for the workshop.

These proceedings are organized as follows:

- Executive summary
- Specific recommendations for both analytical and experimental research needs
- Workshop summary
- Papers presented at the workshop
- List of workshop participants

The recommendations for analytical and experimental research needs were obtained from the working groups. The research topics were prioritized and the level of effort was identified. The priority ranged from 1 (lowest) to 10 (highest).

SUMMARY OF RESEARCH RECOMMENDATIONS RELATED TO CONCRETE/MASONRY REHABILITATION OF LRCF- WORKING GROUP 1

Introduction

A number of strengthening techniques which involve cast-in-place concrete, shotcrete and masonry elements have been studied experimentally and analytically. Some have been used in the strengthening of lightly reinforced concrete (LRC) frame buildings. Some issues associated with concrete and masonry rehabilitations and discussed by the working group include:

- Infill walls, with or without openings, using
    - cast-in-place concrete
    - precast concrete panels or concrete/masonry blocks
    - shotcrete
    - reinforced and unreinforced (in moderate seismic zones) masonry
- CIP or shotcrete shearwalls
- Jacketing
    - column and beam
    - beam-column joint region
- Wingwalls/piers
- Buttresses
- Diaphragms

Some methods, such as column and beam jacketing, have been studied more extensively than others. It was felt that sufficient data was available on jacketing and that guidelines were being developed. Beam-column upgrading was commonly accomplished by the addition of elements made with material other than concrete and was therefore not discussed in detail. In general, it was felt that the available data is still insufficient for the development of guidelines for a number of the strengthening methods listed above. Therefore, more research in these areas was recommended as outlined in the following section.

Some issues discussed in the working group included:

- In the past few earthquakes, the actual performance of many LRC frame structures appeared to be better than that predicted by analyses. This would indicate that these structures probably had more lateral load capacity than was recognized. This raised the need for the development of an accurate, simplified analytical procedure for the evaluation of the lateral resistance capacity of a structure and for criteria on whether or not retrofit is needed. If the results of the evaluation lead to the conclusion that no retrofit is needed, one must necessarily have a very high degree of confidence in the evaluation to be able to follow this conclusion. Accuracy and consistency (from multiple users of similar models) of the results would generate confidence in the analytical procedures.

- If retrofit is needed, then the primary goals should be the enhancement of redundancy and the elimination of the possibility of progressive collapse.

- The retrofit engineering problem is more complex in the Eastern U.S. than in the Western U.S. because of the remote chance of large magnitude earthquakes in the East. Do we need additional ductility in the moderate seismic zones to help a structure through the very infrequent large ground motion event? Load definition for moderate seismic zones needs more attention.

- Compatibility of deformations between the existing frame and the retrofit elements is crucial, and lack of such compatibility will often lead to unsatisfactory performance and even total compromising of the retrofit solution.

- More attention should be given to the effects of the retrofit in changing the potential failure mode of an LRC frame. There is little difference in the hierarchy of failures from one mode to the next and the retrofit can easily suppress one mode and make the structure shift to the next. However, the "next" mode may be less desirable (more catastrophic) than the one that would have resulted if no retrofit had taken place. Again, accurate evaluation methods are needed to predict the performance of the retrofitted structure, particularly its potential failure mode(s).

- There is a need to measure and study the dynamic properties of LRC frames and the influence of non-structural elements on the dynamic properties. It was noted that the vast amount of available data on structural response obtained from instrumented buildings and ground motion from recent earthquakes was underutilized. Such data can increase the understanding of the behavior of LRC frames and the dynamic properties and can help in the validation of the analytical procedures.

  In addition, buildings that are usually instrumented to record strong motions are located in Western U.S. and there is little information on buildings that were designed for wind loads as is typical for structures located in Eastern U.S. These structures are usually taller and not as stiff as those in the Western U.S. It was suggested that further studies be made utilizing the Department of Veteran Affairs' (VA) database which includes evaluation and retrofit of approximately 50 VA hospitals for case studies.

- An important issue dealt with the connection between the existing frames and the added concrete or masonry element. Questions were raised about the necessity for the large number of anchors used in most experimental studies, especially those involving infill walls, to connect the new elements to the existing frames. Further studies on the topic of connections were suggested since connections can be the most costly item in a strengthening project if a large number are required. An alternative connection technique using fewer but larger shear "keys" is currently being studied by Jirsa, et. al.

RECOMMENDED RESEARCH PROGRAMS FOR CONCRETE AND MASONRY REHABILITATIONS

A. ***TITLE:*** Evaluation/Analysis Procedures

PROBLEM STATEMENT: There exists a vast amount of available data on structural response and ground motion from recent earthquakes. However, this data is underutilized. The real world provides the best laboratory in that actual systems including foundations are subjected to real earthquake loads and this advantage should not be lost.

OBJECTIVE: Explain actual behavior of structures experiencing severe earthquakes in terms of analytical results and our general understanding of structural behavior.

TECHNICAL APPROACH: Form a research team of engineers and researchers who would utilize a variety of analysis/evaluation methods of varying sophistication. Conduct several analyses for each selected structure and analyze same frames for different level earthquakes. Compare results to determine consistency of results; understand and explain any differences; define experimental programs to answer questions and to provide information for prediction methods that are more consistent and reliable than present methods.

APPLICABILITY:

Type of Structure: Low = ≤ 4 stories, Medium = 5-8 stories, High = > 8 stories
UBC Seismic Zone: 1 to 4

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	3.75*	3.75	5 (followed by experimental program)		

* 1 man-yr = $200 000

PRIORITY: 10 (1 = Lowest, 10 = Highest). This effort is a prerequisite to put retrofit on a firm basis.

ASSOCIATED RESEARCH NEEDS: Would form basis for decisions leading to necessary experimental programs.

B. *TITLE:* Infill walls

PROBLEM STATEMENT: Infills represent one of the basic methods for upgrading LRC frames. Current design approaches are incomplete and less than consistent. Resolution of the many issues and questions is of urgent priority for retrofitting.

OBJECTIVE: To provide design guidelines for masonry, CIP concrete, and shotcrete (sometimes over existing masonry infills) infills as a retrofit technique for LRC frames. Provide consistent approaches/design conservatism for masonry and concrete infills.

TECHNICAL APPROACH: The following tasks need to be undertaken to accomplish the objectives:

- Synthesize existing experimental data of LRC frames strengthened with infill walls.
- Investigate connections between infill and frames; combine analysis and extensive experiments to fully understand connection behavior.
- Study the influence of openings (size, number, and location) on behavior/ strength/ deformation capacity.
- Determine infill behavior acting with LRC frame, multi-story and multi-bay. Study effects of position of walls and gravity load effects on boundary elements.
- Refine design/assessment methods, establishing drift values (acceptable values) for existing infills
- Develop design guidelines for retrofitting with infills - predict infill performance and guidance for connections between infill and frame.

APPLICABILITY:

Type of Structure: Low = ≤ 4 stories, Medium = 5-8 stories

UBC Seismic Zone: 1 to 4

Method: Increases strength and ductility of structure. Reduces energy dissipation needs.

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	5*	7.5	10	7.5	5

* 1 man-year = $200 000

PRIORITY: 10

ASSOCIATED RESEARCH NEEDS: Connections between new infill and existing concrete.

C. *TITLE:* Precast infill panels for strengthening LRC frames.

PROBLEM STATEMENT: This category is a subset of the previous category, infill walls. However, precast infill panels were discussed in more detail because precast panels show much promise as a material for use in infill wall. This is because they could be manufactured in sizes that are easy to handle and their use would reduce construction time or disruption of the building occupants as the panels are manufactured off-site. However, more tests are needed to verify the feasiblility of the use of precast infill panels and the costs/problems associated with the implementation have to be investigated.

OBJECTIVE: Demonstrate the feasibility and cost of implementation.

TECHNICAL APPROACH: Field application in selected structures. Laboratory studies to determine:

- reinforcement details for providing column tensile strength if overturning produces tension in the columns acting as boundary elements.
- effect of openings in walls.
- effect of details for ducts or utilities through walls.

APPLICABILITY:

Type of Structure: Low = ≤ 4 stories, Medium = 5-8 stories

UBC Seismic Zone: 1 to 4

Method: Increases strength of structure. Reduces damage level.

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	2*	2			

* 1 man-yr = $200 000

PRIORITY: 10

ASSOCIATED RESEARCH NEEDS: Connections between precast panels and between existing frame and precast panels.

D. *TITLE:* Slab-column systems.

PROBLEM STATEMENT: A large portion of LRC frame buildings are constructed with slab-column systems. Research to date on the lateral resistance of such systems is minimal.

OBJECTIVE: Provide data for assessing the performance of existing slab-column systems and the basis for retrofitting slab-column systems.

TECHNICAL APPROACH: A systematic study of the response of slab-column systems to seismic loadings is required. Experimental studies should include:

- 3-D specimens (including existing structures).
- Effects of openings in slabs close to columns.
- Stiffness characteristics.
- Influence of irregular column layouts.
- Multi-bay test specimens.

The performance through large drift values need to be investigated.

APPLICABILITY:

Type of Structure: Low = ≤ 4 stories, Medium = 5-8 stories, High = > 8 stories

UBC Seismic Zone: 1 to 4

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	2.5*	7.5	7.5	5	

* 1 man-yr = $200 000

PRIORITY: 10

ASSOCIATED RESEARCH NEEDS: Incorporate slab-column geometry in other experimental programs.

E. *TITLE:* Diaphragms.

PROBLEM STATEMENT: The addition of collectors, precast members, or diaphragms represent potentially promising techniques for strengthening LRC frames. Currently, existing data is insufficient for demonstrating the workability of this rehabilitation method. A feasibility study is desirable to establish the workability of this technique.

OBJECTIVE: To establish the feasibility of using diaphragms to improve the performance of LRC frame systems and if feasible, to develop design guidelines for the method.

TECHNICAL APPROACH:

- Conduct literature review of available data.
- Develop an experimental program to determine the feasibility of this method.

APPLICABILITY:

Type of Structure: Low = ≤ 4 stories, Medium = 5-8 stories

UBC Seismic Zone: 1 to 4

Method: Increases lateral strength and ductility.

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	1	3	2	1	

PRIORITY: 9

ASSOCIATED RESEARCH NEEDS: Connections between the diaphragms and frame. Analytical studies to determine the global effect of the diaphragms on the behavior of the structure.

F. *TITLE:* Combined Jacketing & Post-Tensioning.

PROBLEM STATEMENT: Combined jacketing and post-tensioning appears to be one of the promising techniques for strengthening LRC frames. At this stage, this technique has not been sufficiently studied. Resolving the many technical questions associated with the use of this technique is a high priority.

OBJECTIVE: To provide data and/or guidelines for the use of combined jacketing and post-tensioning as a retrofit technique for LRC frames.

TECHNICAL APPROACH:

- Conduct experimental studies of LRC columns strengthened by combined jacketing and post-tensioning. Variables include:
  - Column size
  - Amount of post-tensioning
  - Type of post-tensioning
    - Unbonded/bonded
    - Material (steel bars, strands, composite, etc.)
  - Type of jacket (concrete, steel, composite)
- Develop guidelines for strengthening of LRC frames using combined jacketing and post-tensioning.

APPLICABILITY:

Type of Structure: Low = $\leq$ 4 stories, Medium = 5-8 stories, High = > 8 stories

UBC Seismic Zone: 1 to 4

Method: Increases strength and ductility.

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	1	2	2		

PRIORITY: 8

ASSOCIATED RESEARCH NEEDS: Connection/bond of new concrete and existing concrete.

G. *TITLE:* Wingwalls/"wide columns"/Piers.

PROBLEM STATEMENT: Many structures have deep spandrels in the perimeter frames. This results in situations where short, brittle, weak columns are located between strong floors or beams. Such columns are susceptible to shear failure and result in a strong beam-weak column configuration which is undesirable.

There is significant research data available (mostly in Japan) on the use of wingwalls/column widening as a remedial measure. However, the data needs to be synthesized so that a rational approach may be developed on the use of wingwalls/column widening.

OBJECTIVE: Develop practical details, construction methods, and design guidelines to improve the strength and ductility of the short columns through the use of wingwalls/column widening.

TECHNICAL APPROACH: Synthesize the existing data on the use of wingwalls - much of it is from Japan. Based on the review of the existing data, develop an experimental program to fill the required gaps in the existing database. The test specimens should include individual column tests, subassemblies, and multi-bays and/or multi-stories.

APPLICABILITY:

Type of Structure: Low $= \leq 4$ stories, Medium = 5-8 stories, High $= > 8$ stories

UBC Seismic Zone: 1 to 4

Method: Increases strength and ductility of structure.

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	1*	3	3	1.5	

* 1 man-yr = $200 000

PRIORITY: 7

ASSOCIATED RESEARCH NEEDS: Connection of new concrete to existing concrete.

H. ***TITLE:*** Buttresses (Addition of a new concrete wall).

PROBLEM STATEMENT: The success of using buttresses to enhance the lateral strength of LRC frames depends largely on the load transfer mechanism from the existing structure to the buttresses. The interaction between the existing structure and the added buttresses is influenced by the connections and presently is not well understood.

OBJECTIVE: To develop design guidelines for the design of buttresses as a means to improve seismic performance of LRC frames.

TECHNICAL APPROACH: Conduct both experimental and analytical studies to identify critical design parameters affecting the performance of LRC frames strengthened by buttresses.

APPLICABILITY:

Type of Structure: Low = ≤ 4 stories, Medium = 5-8 stories, High = > 8 stories

UBC Seismic Zone: 1 to 4

Method: Increase structural stiffness.

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	1	2	1		

PRIORITY: 5

SUMMARY OF RESEARCH RECOMMENDATIONS RELATED TO STEEL REHABILITATION OF LRCF- WORKING GROUP 2

Introduction

Steel elements are common materials used in the rehabilitation of lightly reinforced concrete frames. Several methods of rehabilitation involving the use of steel elements are currently being used or studied. The main uses of steel in rehabilitation schemes are:

- jacketing of columns or beams
  - full with steel plates
  - partial with steel plates
  - other methods such as laced sections, strips, etc.
- beam-to-column jacketing and joint jacketing to possibly upgrade the frame to a moment resisting frame
- bracing systems that act compositely with existing frame
  - concentric using common steel shapes or tendons
  - eccentric - horizontal or vertical shear links
- addition of a new moment frame to the existing concrete frame

In all of the above methods, the connections between the steel elements and the existing concrete frame are of great importance. These connections could be direct connections such as bolts, anchors, dowels, etc., or they could be indirect connections whereby the steel infill is connected to the frame using cast-in-place or wet-connections. Existing data on connections are usually provided by the manufacturer. Since no standard test procedures have been identified, comparison of the data/properties from different manufacturers is difficult. Therefore, standard test protocols to evaluate the connectors have to be established.

The availability of the test data, the assembly and synthesis of the data, and the availability of design guides were discussed for each of the methods listed above. A common need for all the methods discussed is the compilation and digestion of the available data. In some cases, it was felt that sufficient data was available from various research facilities. However, the test results and findings were not accumulated, processed and correlated to develop rational design guides. It was also felt that even if insufficient data was available, a catalog of the available data/results would be of assistance to designers. Another issue common to all the rehabilitation methods is the need to better understand the load path and the transfer of the loads from existing elements to the new elements.

In addition, the group agreed that any experimental and analytical research recommended should be conceived and tailored to provide information that is readily usable in design guidelines. Also, the objective of the research should be the generation of results that could be used in the development of a common design methodology rather than in the development of narrowly focused design procedures. To achieve this goal, the research results should include information on the stiffness, yield levels, and ductility of the rehabilitated frame. Also, limit states and acceptability criteria for various performance objectives should be identified.

RECOMMENDED RESEARCH PROGRAMS FOR REHABILITATION METHODS INVOLVING STEEL ELEMENTS - WORKING GROUP 2

A. *TITLE:* Connectors for attaching new steel elements to existing concrete.

PROBLEM STATEMENT: There is little test data reported in the literature on connectors in concrete and what is available is usually provided by the manufacturer. Therefore, it is important for standard test procedures for the evaluation of connectors to be developed.

Surface preparation standards are available for new construction only. These procedures may be inappropriate or very costly for use in existing construction.

OBJECTIVE: Provide consistent data on strength and deformability of connections that address surface preparation, group action, and rate of loading.

TECHNICAL APPROACH:

- Develop a catalog of available data on connections.
- Develop a catalog of data gaps.
- Develop a test protocol.
  - Number of cycles of loading
  - Displacement limits
  - Rate of loading
  - Type of loading - shear, tension, combined shear and tension, etc.
  - Acceptance criteria
- Test parameters:
  - Single connectors
  - Group of connectors
  - Edge distance
  - Spacing of connectors
  - Surface preparation
  - Indirect connectors
  - New innovative connectors / Advanced materials
  - Interaction of forces (axial, shear, flexure)
- Test subassemblages to determine the effect of relative stiffness, load redistribution, etc.
- Develop a "handbook" on connections.

APPLICABILITY:

Type of Structure: Low $= \leq 4$ stories, Medium = 5-8 stories, High $= > 8$ stories

UBC Seismic Zone: 1 to 4

Method: Increases strength and ductility of structure. Reduces demand, energy dissipation needs and damage level.

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
Item No.	1 - 3	4	5	6	

PRIORITY: 10. This data is critically needed and research budget required is relatively low.

ASSOCIATED RESEARCH NEEDS: None

COMMENTS: The same data is required for new concrete to existing concrete connections.

B. *TITLE:* General issues for enhancing the practical use of research data.

PROBLEM STATEMENT: Reporting of experimental data is not focused towards direct applications by users. Data on material properties and acceptability criteria are not provided in a form that can be used by the existing codes/guidelines. Possible risks or pitfalls of experimentally-tested rehabilitation methods are not mentioned in most reports.

OBJECTIVE: Streamline application of research results into rehabilitation guidelines. Encourage researchers to include in their research reports cautionary items that designers should be aware of, such as the possible change in failure mode due to the rehabilitation strategy, or deformation incompatibility of the upgraded elements with the existing frame.

TECHNICAL APPROACH:

- Focus research on data needs of ATC 33.
- Present results of research which address parameters that are needed for modeling and acceptance criteria.
- Include in each research reports, information on the shift of the "weak" link in a rehabilitated structure when applicable.

APPLICABILITY:

Type of Structure: Low = $\leq$ 4 stories, Medium = 5-8 stories, High = $>$ 8 stories

UBC Seismic Zone: 1 to 4

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	Not applicable				

PRIORITY: 8

ASSOCIATED RESEARCH NEEDS: None

C. *TITLE:* Steel Jacketing

PROBLEM STATEMENT: Reinforced concrete gravity frames often suffer from inadequate dowel laps, development lengths, or confinement to prevent bar buckling and inadequate shear capacity to resist lateral loads. Substantial test data exists on the rehabilitation method involving steel jackets to correct these deficiencies. However, there are no comprehensive documents or preliminary guidelines that suggest appropriate retrofit strategies for use by designers.

OBJECTIVE: Develop guidelines for the use of steel jacketing for rehabilitation of gravity frames to remedy the deficiencies of these frames as stated above.

TECHNICAL APPROACH:

- Collect and organize data with respect to RC column and beam jacketing.
- Analyze data and categorize data by the deficiency addressed.
- Prepare preliminary design guidelines for use by designers.
- Identify gaps in existing data or where further research is needed.

APPLICABILITY:

Type of Structure: Low = $\leq$ 4 stories, Medium = 5-8 stories, High = > 8 stories

UBC Seismic Zone: 1 to 4

Method: Increases strength and ductility of structure.

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	1	1			

PRIORITY: 9

ASSOCIATED RESEARCH NEEDS: Additional jacketing schemes addressing cost and ease of placement.

D. *TITLE:* Retrofit of beam-to-column joints using external steel elements.

PROBLEM STATEMENT: Limited data exist for tests investigating improvement in joint shear capacity and anchorage of bars in the joint by means of attachment of external steel elements. Little guidance in the form of design recommendations for retrofitting joints has been provided.

For some systems and seismic zones, such as low-rise buildings in low seismic zones, improving the behavior of the joint region may not be necessary.

OBJECTIVE: Determine when discontinuous positive moment reinforcement in beams, lack of confinement of hooks in exterior joints, and insufficient shear strength and/or confinement in joints are acceptable or unacceptable. When strength and ductility must be enhanced, what methods are recommended?

TECHNICAL APPROACH: Sufficient data describing the behavior of non-ductile beam-column joints is available. This behavioral information should be analyzed to assess the limits of deformability of these connections. Overall retrofit systems that do no require retrofit of joints should be identified.

Additional tests investigating methods for improving deformability of joints, shear strength, anchorage of beam bottom bars, and confinement of hooked-bar anchorages in exterior joints should be conducted. Development of design recommendations for these methods should follow.

APPLICABILITY:

Type of Structure: Low = ≤ 4 stories, Medium = 5-8 stories, High = > 8 stories

UBC Seismic Zone: 1 to 4

Method: Increases strength and ductility of structure. Reduces damage level.

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	1	3	3	1	

PRIORITY: 8

ASSOCIATED RESEARCH NEEDS: Connection of steel to existing RC.

E. *TITLE:* Seismic upgrading using complete steel frames.

PROBLEM STATEMENT: The seismic behavior of reinforced concrete frame buildings can be improved by adding a steel frame as a new and complete lateral force resisting system. The design of such new frames is governed by available design methods for new buildings. However, reliable procedures for the design of connections between the new steel frame and the existing concrete building do not exist. This presents a major obstacle in the standardization of this rehabilitation procedure. Another obstacle is the deformation compatibility between the steel frame and the concrete frame.

OBJECTIVE: Develop reliable procedures for designing connections of new frames to existing concrete.

TECHNICAL APPROACH: See approach for Connections.

APPLICABILITY:

Type of Structure: Low = ≤ 4 stories, Medium = 5-8 stories, High = > 8 stories

UBC Seismic Zone: 1 to 4

Method: Increases strength and ductility of structure.

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	See effort for Connections.				

PRIORITY: 8

ASSOCIATED RESEARCH NEEDS: Connections between steel elements and RC. Deformation compatibility of existing RC frame and new steel frame.

F. *TITLE:* Assessing deformation compatibility in rehabilitated buildings.

PROBLEM STATEMENT: When the seismically weak links in a building are upgraded, the overall behavior of the structure is most likely changed. In order for the rehabilitation to be successful, building sections which have not been upgraded (and which may not be part of the lateral force resisting system) must be able to withstand the deformations that are imposed on them by the newly rehabilitated elements without causing unacceptable consequences.

A complete rehabilitation design guideline must include a standard recommended procedure for checking the compatibility of the existing and new structural elements. An analysis procedure is currently being developed by the FEMA Seismic Rehabilitation Guidelines and Commentary project, ATC 33, but the completeness of the procedure is limited by the availability of experimental data.

OBJECTIVE: To identify and perform experimental research needed to complete and improve the analysis procedure for checking the compatibility of existing and new structural elements under seismic loads.

TECHNICAL APPROACH:

- Seek input from ATC 33 and others on gaps in the experimental database.
- Develop prioritized list of research projects.
- Perform research.
- Feed results back to ATC 33 for incorporation into the analysis procedures.

APPLICABILITY:

Type of Structure: Low = ≤ 4 stories, Medium = 5-8 stories, High = > 8 stories

UBC Seismic Zone: 1 to 4

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	1	3-10	3-10	3-10	3-10

PRIORITY: 7

ASSOCIATED RESEARCH NEEDS: None

G. *TITLE:* Bracing systems in composite action with lightly reinforced concrete frames.

PROBLEM STATEMENT: Steel bracing systems, concentric and eccentric, have been very popular in enhancing the stiffness, strength, and ductility of a structure. The use of structural shapes as braces are more common, but prestressed diagonal tendons are also used for stiffness and strength enhancement. Some test and analytical data and recommended procedures for the design and analysis of members exist. However, there is insufficient information on the properties of the elements and load transfer mechanisms between steel and RC elements.

OBJECTIVE: Develop element properties (strength, stiffness, ductility) that are required for the inelastic analysis of the combined system. Develop design procedures to determine the required forces in the design of connectors between steel elements and RC elements.

TECHNICAL APPROACH: There exists a lot of data on the behavior of steel frames and eccentric braces, but not on the behavior of a combined system, concrete frame and steel frame/ eccentric braces. An experimental and analytical effort of carefully selected subassemblages subjected to realistic loading and deformation conditions is needed. The subassemblages should represent both concentric and eccentric bracing systems.

APPLICABILITY:

Type of Structure: Low = ≤ 4 stories, Medium = 5-8 stories, High = > 8 stories

UBC Seismic Zone: 1 to 4

Method: Increases strength and ductility of structure. Reduces damage level.

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	1	0.5	0.5		

PRIORITY: 6

ASSOCIATED RESEARCH NEEDS: Behavior of connections. Analytical modeling properties for system analysis.

SUMMARY OF RESEARCH RECOMMENDATIONS RELATED TO COMPOSITES AND DAMPING SYSTEMS REHABILITATION OF LRCF- WORKING GROUP 3

Introduction

Compared with the rehabilitation methods involving concrete/masonry or steel, rehabilitation methods involving composites and damping systems are considered innovative or newer methods. Therefore, there is less data available for these methods than for more conventional methods discussed in the previous two sections.

Several studies have been conducted on the rehabilitation of bridge piers with composite wraps. Very few studies exist on the use of composites for the rehabilitation of buildings. Material data for composites are available from the manufacturers but information on additional properties such as the stiffness and the durability of the material from exposure to stresses such as fire, ultra-violet radiation, high/low temperatures, for example, are necessary if it were to be used in a rehabilitation scheme.

Feasible applications for composite materials include the replacement of steel elements with composite materials in the rehabilitation scheme; use of composite plies in lieu of steel plates and composite wraps instead of steel jackets, etc. The use of composite laminates to limit damage is a viable concept as in the case of unreinforced masonry walls subjected to out-of-plane loading. As with the other rehabilitation methods, the "connection" between the composite material and the concrete is an important issue. The advantages of using composites instead of steel elements are composites are much lighter, on the order of 10 times, and the attachment/ application of the composite material is much easier. However, much research, both experimental and analytical, is required to build the necessary database before the development of guidelines is possible.

As there are no guidelines for the use of composites to rehabilitate LRC frames, it was recommended that peer review and proof testing be conducted to verify the method and concept prior to use.

The second topic of discussion was damping devices. Prior to the workshop, a decision was made to exclude base isolation systems from the discussion due to limited time available in this workshop. Damping devices include hysteretic devices, viscous devices, and mass dampers. As the name implies, the function of the damping device is to increase the damping of the structure. However, these devices may also increase the strength and stiffness of the structure. The development of this technology is further along than that for composites. Some damping devices are in use in the U.S. and abroad.

One of the major obstacles to the widespread use of this technology is the sophisticated and costly analyses that are needed to determine the behavior of a structure rehabilitated with damping devices. Therefore, simplified analytical procedures need to be developed. The design procedure will have to be able to link the properties of the damping device to the performance of the structure to allow for the prediction of the structural behavior under seismic, wind, or gravity loads. In addition to developing simplified design procedures, a standard evaluation procedure is needed to quantify the properties of the different damping devices.

In the experimental area, the durability, reliability, environmental effects, maintenance, and connections of the damping devices to the existing frames have to be studied. It was felt that sub-assemblage tests should be conducted followed by case studies. These case studies should involve typical RC structures in all seismic zones. Also, structures that are federally owned or leased are good candidates for case studies due in part to liability issues involved in a privately owned building.

The following section lists tasks and recommended research that would increase the knowledge base in the area of composite and damping systems.

RECOMMENDED RESEARCH PROGRAMS FOR REHABILITATION METHODS INVOLVING COMPOSITES

A. *TITLE:* Catalog composite materials for structural applications.

PROBLEM STATEMENT: No comprehensive list of materials is currently available for use by designers. Material and material properties may be obtained from the manufacturer.

OBJECTIVE: Create a state-of-the-art database of materials that are commercially available and their properties - mechanical, constructibility/ease of use, cost, availability, packaging, durability, etc.

TECHNICAL APPROACH: Conduct a literature survey, contact manufacturers, contact professional societies such as ASTM, ACI, etc, and hold workshop(s) for manufacturers and researchers.

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	1				

PRIORITY: 10

ASSOCIATED RESEARCH NEEDS: Explore research studies involving durability, longevity, and effects of the environment.

B. *TITLE:* Experimental tests of composite materials for rehabilitation.

PROBLEM STATEMENT: Some of the common deficiencies of concrete frame structures (as listed in "Structural Applications of Rehabilitation Using Composites") appear to be candidates for correction with externally applied composite materials. The materials may be advantageous due to their light weight and small thickness requirements. Most of the concepts require experimental verification for a variety of real world variations in configurations and materials. Work to date is somewhat limited, especially for application with respect to seismic rehabilitation.

OBJECTIVE: Prove (or disprove) feasibility of various applications for rehabilitation, i.e., determine which methods work and which do not and why. Develop the basis for analytical models and methods for design.

TECHNICAL APPROACH: Test physical applications including:

	Effort (man-yr)	Priority
• Wraps for compression ductility	2	8
• Wraps for deficient rebar laps	2	8
• Wraps for shear strength	3	8
• "Active" wraps with initial confining pressure	2	8
• Skins for walls improving out-of-plane behavior	3	8
• Improvement of joints - beam-column upgrade	4	8
• Diaphragm chords	3	7
• Wraps at ends of walls	2	7
• Precast joint strengthening (providing continuity across joints)	4	6
• Fiber cables as diagonal bracing	3	5
• Crack repairs	2	5
• Others to be identified by parallel research		

All the preceding should be studied with a variation of parameters such as concrete strength, type of fiber, type of resin, thickness of wraps, physical configuration of members, joints, and frames.

APPLICABILITY:

<u>Type of Structure:</u> Low = $\leq$ 4 stories, Medium = 5-8 stories, High = > 8 stories

<u>UBC Seismic Zone:</u> 1 to 4

<u>Method:</u> Increases strength and ductility of structure. Composite materials are usually brittle but it can increase the ductility of an element by preventing

premature brittle failure. In some cases, reduction in damage level is possible.

EFFORT (MAN-YEAR): SEE TECHNICAL APPROACH FOR BREAK DOWN

PRIORITY: 10. Although no single item is rated highly (10), priority is very high to do some work in this area which shows promise.

ASSOCIATED RESEARCH NEEDS: Systematic analysis to define most likely beneficial applications.

C. *TITLE:* Analytical evaluation of component strength and ductility requirements for structural system retrofit using composites.

PROBLEM STATEMENT: Non-ductile RC frames may pose threats to their occupants in the event of a major earthquake. To develop effective retrofit schemes and design guidelines, the demand imposed on the retrofit and the benefit of implementing the retrofit must be established. This information will provide a basis for developing subsequent experimental programs on components that will involve proof-testing of the retrofit schemes to see if the "resistance" is adequate to meet the demand.

OBJECTIVE: To evaluate the effects of component strength and ductility enhancement on structural system performance.

TECHNICAL APPROACH: Develop analytical models of generic LRC systems. Consider structural system and component type as well as building height as parameters.

- Perform nonlinear static push-over analysis of as-built generic systems (Task 1).
- Nonlinear static push-over analysis of retrofitted generic systems (Task 2).
- Compare the results of 1) and 2) to assess the demand and effectiveness of the retrofit.
- Conduct experimental tests of frames rehabilitated using composites to verify analytical models.
- Repeat the Tasks 1 and 2 using nonlinear time history analysis and a suite of earthquake records (this becomes tasks 3 and 4).

Generic RC systems: Moment resisting frames, Shear wall systems, Infill URM frames

APPLICABILITY:

<u>Type of Structure:</u> Low = ≤ 4 stories, Medium = 5-8 stories, High = > 8 stories

<u>UBC Seismic Zone:</u> 2 to 4

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	2	2	Exp. test	2	2

PRIORITY: 10. Tasks 1 and 2 are high priority and very essential. Tasks 3 and 4 are not as essential and should be done after some experimental data on retrofit becomes available to quantify component force-deformation response.

COMMENTS: Benchmark analysis problems need to be identified. Effectiveness implies evaluating the performance level of the structural system with retrofitted components.

ASSOCIATED RESEARCH NEEDS:

1. Development of retrofit schemes with laboratory proof-testing in order to obtain component force-deformation characterization.

2. Develop simplified analytical tools for engineering practitioners to predict retrofitted component and system performance.

D. *TITLE:* Structural applications of rehabilitation using composites.

PROBLEM STATEMENT: Need to identify and develop techniques to solve major areas of deficiencies of frames:

- Shear sensitive sections - failures
- Lack of continuous reinforcement
- Widely spaced ties and inadequate confinement
- Poor positioning of lap splices and confinement
- Overall frame proportions resulting in strong beam/weak column concept

OBJECTIVE:

- Identify the most promising applications to solve deficiencies (wrapping, coating, reinforcing, hybrid of materials, etc.).

- Screen suggested techniques using analytical methods and identify further evaluation needs - analytical and experimental.

- Develop a program for research - analytical and experimental.

TECHNICAL APPROACH:

- Create a committee for planning and reviewing current status. Involve practicing engineers, researchers, and manufacturers.
- Select categories of buildings (building height, type, and seismic zone).
- Evaluate status of current solutions vs. applications.
- Identify areas that need further in-depth evaluation - experimental and analytical.

APPLICABILITY:

Type of Structure: Low = $\leq$ 4 stories, Medium = 5-8 stories, High = $>$ 8 stories

UBC Seismic Zone: 1 to 4

Method: Increases strength and ductility of structure. Reduces energy dissipation needs.

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	1	0.5	0.5		

PRIORITY: 9

ASSOCIATED RESEARCH NEEDS: Evaluation of material properties. Analytical tools for integral evaluation.

E. *TITLE:* Retrofit in the emergency environment using composites.

PROBLEM STATEMENT: There is a need to prove the concept of composites in emergency retrofit of government buildings/lifelines. The principal issues in emergency response and recovery are damage assessment (i.e. red, yellow, green tagged buildings) and rapid restoration of community services to "normal". A slow recovery increases indirect losses which are 2 to 10 times direct losses.

OBJECTIVE: To create a knowledge base and procedures for the use of composites for retrofit following a damaging earthquake and subsequent aftershocks.

TECHNICAL APPROACH: As there is no knowledge base currently, the first task is to develop a plan for deploying technical experts and composite materials as part of the post earthquake investigations/emergency response. This plan could be developed by conducting a workshop. A priori agreements can be reached for applications on selected types of government buildings and/or lifelines. The second step involves 5 to 10 field implementations in the U.S. to gain experience. The third step is to evaluate the results using a combination of experimental and analytical data. Based on the third step, the fourth step is to refine the plan to make it more efficient. The fifth step is to transfer the technology nationally and internationally.

APPLICABILITY:

Type of Structure: Low = $\leq$ 4 stories, Medium = 5-8 stories, High = $>$ 8 stories

UBC Seismic Zone: 4. The most likely occurrences will be in California, but plan for other seismic zones as well.

Method: Risk management

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	2	3	3	3	2

PRIORITY: 7

COMMENTS: The economic payoff could be very significant.

ASSOCIATED RESEARCH NEEDS: Requires ongoing experimental and analytical studies.

RECOMMENDED RESEARCH PROGRAMS FOR REHABILITATION METHODS INVOLVING DAMPING SYSTEMS

F. *TITLE:* Development of simplified design and evaluation techniques.

PROBLEM STATEMENT: Current evaluation techniques are complex and require time history analyses. A simple engineering technique is needed to promote this new technology. Preliminary studies showed that the push-over technique and composite spectra are viable evaluation techniques. The push-over and composite spectra techniques are approximate approaches and their feasibility and limitations need to be thoroughly investigated.

OBJECTIVE: Develop rapid, simple, and unified procedures for engineers/architects to determine or quantify the influence of dampers.

TECHNICAL APPROACH:

- Develop several benchmark problems from full-scale case studies.
- Develop a parametric study for each structural type involving:
  - Type of damping system
    - viscous systems
      - solid
      - fluid
    - hysteretic systems
      - friction type
      - yielding elements
    - mass or tuned dampers
  - Hazard levels
  - Variability of structural parameters
  - Performance levels
- Use time history analyses to determine limitations for simplified methods (push-over analyses and composite spectra).
- Iteratively develop the simplified technique and its limitations
- Verify simplified technique by comparing results of the simplified technique obtained from various researchers for the same model.
- Integrate program with regulatory bodies to develop guidelines.
- Integrate evaluation groups - researchers, engineers, manufacturers, regulatory officials.

APPLICABILITY:

Type of Structure: Low = ≤ 4 stories, Medium = 5-8 stories, High = > 8 stories

UBC Seismic Zone: 1 to 4

EFFORT (MAN-YEAR):	Yr. 1	Yr. 2	Yr. 3	Yr. 4	Yr. 5
	6	5	2		

PRIORITY: 10

ASSOCIATED RESEARCH NEEDS: Integral analytical modeling of structures using nonlinear/inelastic techniques validated by experiments.

G. *TITLE:* Evaluation of the efficiency of the retrofit of structural systems using damping and energy dissipating devices.

PROBLEM STATEMENT: There is a need to evaluate the effectiveness of the many different devices on retrofitted structural system performance. In addition, design guidelines for their use in enhancing the structural system performance is needed.

OBJECTIVE: To evaluate the effectiveness of various damping and energy dissipating devices in retrofitting of LRC structures to improve their seismic performance and to mitigate earthquake hazards.

TECHNICAL APPROACH: Develop analytical models of generic LRC structural systems - moment resisting frames, shear wall systems, infilled URM frames. Consider structural system, component type, and building height as parameters.

- Perform nonlinear static push-over analyses (Task 1).
- Perform nonlinear time-history analyses using experimentally obtained force-deformation relationships. Consider the different devices, hysteretic, viscous, and mass dampers, as a parameter. (Task 2)
- Perform nonlinear static push-over analysis on the retrofitted structures (Task 3).
- Perform nonlinear time-history analyses of the retrofitted structures (Task 4).
- Compare the results of Tasks 1 and 3 and 2 and 4 to assess the effectiveness of the different retrofit schemes.
- Develop design guidelines for using damping devices (Task 5).

APPLICABILITY:

Type of Structure: Low = ≤ 4 stories, Medium = 5-8 stories, High = > 8 stories

UBC Seismic Zone: 1 to 4

Method: Increases strength and ductility of structure. Reduces energy dissipation needs by conventional structural elements.

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	2	1	1	1	2

PRIORITY: 10

ASSOCIATED RESEARCH NEEDS: Development of simplified analytical tools for evaluating performance of retrofitted systems and incorporating these tools into the design recommendations.

H. *TITLE:* Develop guidelines for the use of energy dissipating devices (EDD) for field implementation.

PROBLEM STATEMENT: Various issues have arisen from the implementation of energy dissipating devices in the U.S. Such issues must be resolved for the three types of devices of interest: hysteretic (yielding element or friction type), viscous (solid and fluid), and tuned systems. Guidelines are required for the use of the EDD in representative structures in various seismic zones in the U.S. Case studies of structures employing EDD are needed to provide information on the actual performance of the EDD and to calibrate the analytical procedures.

OBJECTIVE: Develop guidelines for the use of the three types of EDD: hysteretic, viscous, and tuned systems. These guidelines should apply to representative structures in all seismic zones.

TECHNICAL APPROACH:

- Develop inventory of all devices available.
- Develop guidelines for qualification of devices.
- Develop guidelines for maintenance, conditions of use (See "Life cycle evaluation of EDD).
- Establish analytical models for various EDD.
- Identify and resolve issues arising from implementation.
- Conduct case studies
  - Involve manufacturers and engineers.
  - Determine procedures to monitor, record, and analyze the structural response.
  - Sample different seismic zones and representative structures.
  - Use information from case studies to refine the above guidelines as needed.

APPLICABILITY:

Type of Structure: Low = $\leq$ 4 stories, Medium = 5-8 stories, High = > 8 stories

UBC Seismic Zone: 1 to 4

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	3	2	2	1	1

PRIORITY: 8

ASSOCIATED RESEARCH NEEDS: "Life cycle" considerations (durability, longevity, effects of environment), analytical models.

I. *TITLE:* Life cycle evaluation of energy dissipating devices.

PROBLEM STATEMENT: Information on the impact of environmental factors including temperature, UV, etc. is incomplete. Maintenance and operational requirements must be established for optimal performance and reliability. Cost benefit data needs to be developed on a uniform basis.

OBJECTIVE: Develop guidelines for establishing life cycle performance qualification criteria.

TECHNICAL APPROACH: Compile existing information. Identify issues/criteria which are currently not available for devices. Establish testing procedures for evaluating life cycle factors including:

- performance reliability of devices (durability)
- accelerated environmental effects (longevity)
- operation and maintenance costs

Perform sample cost/benefit studies for devices based on design/construction and life cycle costs relative to different performance objectives.

APPLICABILITY:

Type of Structure: Low = ≤ 4 stories, Medium = 5-8 stories, High = > 8 stories

UBC Seismic Zone: 1 to 4

EFFORT (MAN-YEAR):	**Yr. 1**	**Yr. 2**	**Yr. 3**	**Yr. 4**	**Yr. 5**
	0.5	0.5			

PRIORITY: 8

ASSOCIATED RESEARCH NEEDS: Analytical studies related to performance for different rehabilitation objectives.

WORKSHOP SUMMARY

Discussions in the working groups focused on the state-of-knowledge in the rehabilitation of lightly reinforced concrete or gravity frames and on the identification of additional information necessary to develop practical design guides. The rehabilitation methods discussed involved the use of concrete, masonry, or steel elements, composites and damping systems.

Participants identified what information was available and if the data is not available, what additional data was needed. In many cases, the collection and assimilation of all the available data on a particular rehabilitation method/material was felt to be beneficial even if there wasn't sufficient data to develop guidelines.

In addition to the specific research areas or tasks identified in each of the working groups, some common issues that arose in the discussions and that were felt to be of particular importance by all three working groups were:

- The designer/engineer has to account for the deformation compatibility between the new elements and the existing frame. This is an important issue because the new elements and the existing elements usually have different stiffnesses. Lack of compatibility will often lead to unsatisfactory performance.

- Connections between the new elements (concrete, masonry, steel, composites, damping systems, etc.) and the existing frame are very important. The success of any rehabilitation method is greatly influenced by how well the connections perform their intended purpose of load transfer. There is an economic incentive in being able to rationally determine the required amount of connections instead of installing more than are necessary to ensure the desired objective/behavior.

- There is a need to develop simplified analytical methods/procedures to evaluate the performance of the existing structure and the rehabilitated structure. The availability of such procedures will allow for the determination of whether or not retrofit is needed and the benefits of the retrofit. The analytical procedures must yield accurate and consistent results for them to be utilized with any confidence.

- By selecting a particular rehabilitation method, the designer/engineer has to be cognizant of how the failure mode changes in the rehabilitated structure, both locally and globally. A change in failure mode caused by the shifting of the weak link to an undesirable or unexpected location could have disastrous consequences. Again, analytical procedures to predict the structural performance, particularly the failure mode(s), is of key importance.

- The additional demand on the foundation caused by the rehabilitation of a structure has to be investigated.

- Research data and findings are available for the various rehabilitation methods, more in some area than in others. However, there is a need for a group/agency to assemble and to integrate the data and to present the data in a practical format (design guidelines/procedures, graphs, tables, etc.) that can be easily used by the design profession. However, there was little discussion as to the assignment of a group/agency for this responsibility.

- Use available data from buildings that have undergone an earthquake and data from past earthquakes to better understand the performance of the buildings that have sustained damage and those that have not. The data should be used to calibrate the available analytical programs to better predict the performance of a structure.

In addition, experimental and analytical studies were recommended by all groups. However, for ease of comparison of test results between the various experimental programs, a standard test protocol is needed. This would include specifying a displacement/load history, the number of cycles at each displacement/load level, specimen scale, and definitions of terms such as stiffness, yielding, failure, etc. For the inelastic dynamic analyses, a standard set of input motion obtained from various earthquakes is required.

SEISMIC REHABILITATION OF NON-DUCTILE REINFORCED CONCRETE FRAMES -- A Summary of Issues, Methods, and Needs

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ABSTRACT

This paper examines the issues pertaining to selection of retrofit strategies for existing reinforced concrete framed buildings designed primarily for gravity loads. Selected retrofit methods based on concrete and masonry components are reviewed. Loading protocols for experimental study of retrofit performance are presented, and suggestions are made on format and contents of retrofit guidelines.

1.0 INTRODUCTION

This paper is directed to the broad topic of rehabilitation (upgrading, retrofit, strengthening, toughening, etc.) of reinforced concrete (RC) frames to ensure that they can resist lateral forces substantially higher than those used in the original design of the structure. This type of RC frame has been described by many different names -- non-ductile, lightly reinforced, and gravity load design. Whatever the terminology, the common denominators for all such frames include (a) lack of continuity of positive moment reinforcement in the joint regions, (b) widely spaced ties and inadequate confinement of concrete in the columns and joints, (c) poor positioning of lap splices in column reinforcement, and (d) overall frame proportions which may result in a strong beam-weak column collapse mechanism under severe reversing lateral loads.

As a matter of convenience, the process of rehabilitation and retrofit will be abbreviated as R/R in most places in this paper.

2.0 PURPOSE AND SCOPE

This paper is intended to:

1. Identify and review the primary issues involved in the decision-making process for retrofitting or rehabilitating a reinforced concrete frame building,
2. Review selected methods of rehabilitation/retrofit involving concrete, prestressed concrete, and masonry, with some coverage related only to research results and other coverage directed to specific methods as used in actual retrofitting projects.
3. Begin the process of defining research needs and future directions for research,

4. Discuss guidelines required for practicing engineers and make suggestions on guideline development approaches.

The paper is written specifically to serve as a basic introduction to this 2-day NIST Workshop on Seismic Rehabilitation. As part of the coverage of the four topics outlined above, the paper is written to provoke questions and to help define a consistent structure for the follow-up sessions where specific R/R methods, and their associated research and development needs, will be discussed in detail.

In keeping with the overall purpose and scope of the NIST Workshop, the paper does not attempt to provide any significant coverage of issues and methods related to the important topic of repair of frames already damaged by seismic loadings. Nor does it cover retrofit of non-ductile flat-plate structures susceptible to punching shear failures when subjected to seismic action.

The workshop sessions are divided into three distinct categories of retrofit -- R/R with concrete/masonry, R/R with steel, and R/R with composites and damping systems. It should be remembered that practical retrofit schemes for a given building may well combine techniques from two or even three R/R categories.

3.0 ISSUES INVOLVED IN RETROFIT & REHABILITATION

3.1 Definition of the Issues

A multiplicity of issues is encountered in deciding on R/R for any given RC frame building. Considerable innovative engineering is usually required. This engineering needs to be carried out within the context of a broad set of issues and questions. Some issues are (or will eventually be) reasonably well-defined in published guidelines and codes of practice. Others cannot (and should not) be codified or specified and accordingly will require important decisions on a building-by-building basis.

The following listing of issues and questions to be considered is given here to help stimulate critical thinking about the entire retrofit process. While many of these issues fall considerably beyond the scope of this NIST Workshop (and we are obviously not going to attempt to resolve them) stating them "upfront" may be productive in helping to generate an appropriately broad perspective for our detailed discussions over the next two days. With this qualifying statement in mind, the listing is given below. Note that limited answers are given to items 1, 4, and 6; at least this level of discussion could have been given to all other points.

1. Just what is the primary area of concern with lightly reinforced concrete (LRC) frames? What is the effect of non-ductile details on building performance and how do these effects vary with building height? Partial answers are provided by Sause, Pessiki, Kurama, and Wu [1994], who developed a new model for representing the

pullout moment-rotation behavior of the discontinuous embedded positive reinforcement used in beams in LRC frames, as based on the extensive experimental results from several Cornell NCEER-sponsored studies (Beres, White, and Gergely [1992]). This same model can also be modified to capture the non-ductile moment-curvature behavior of column end regions. Sause et al incorporated the model into DRAIN-2DX for use in the analysis of typical 3-story and 12-story LRC building frames, with each building being 5 bays by 5 bays in plan.The buildings were designed by two different methods -- working stress design, WSD (ACI 318-56 and detailing manuals in use at that time) and ultimate strength design, USD (ACI 318-63 and corresponding detailing manuals).

Inelastic static analyses of the two structures were carried out for combined gravity and lateral loads, with two different lateral load profiles -- triangular and rectangular. Results are summarized in Table 1. The author's conclusions are as follows:

> "The seismic behavior of the prototype structures was controlled by the non-ductile behavior of the critical regions. These critical regions require retrofit to improve the seismic behavior of the prototypes. Critical column end regions must be retrofit to increase the global ductility capacity. However, such a retrofit would not significantly increase the base shear capacity of the 12-story structure; pullout regions at the beam-column joints must be retrofit to increase the base shear capacity of this structure. Possible non-ductile moment/curvature behavior in the beams is not considered to be critical to the base shear or global ductility capacity of the prototypes. However, if the pullout regions are retrofit to increase their ultimate moment/rotation capacity, the other regions in the beams should be carefully evaluated."

2. What level of evaluation of the existing RC frame building is needed in making the critical decision on whether or not retrofit is needed? What level of analysis should be used? How do we best include the effects of structural, "weak" structural, and non-structural elements such as walls, partitions, staircases, etc.? Is non-linear pushover analysis a good approach? Will time history analysis be used with any regularity?

3. Can we expect the "average" structural engineer to do retrofit studies, or are we aiming our end product at a small group of "structural specialists"? Just who is the audience for a new set of guidelines for retrofit of LRC frames (or any other type of structure)? How do we best transfer new knowledge to engineers, to building officials, and to other interested parties? What is the obligation of the engineer in educating the owner about the many subtleties of R/R?

4. Which characteristics of the frame need improvement -- strength, stiffness, or ductility? What are the trade-offs to be made? An appreciation of the redesign

strategies of Jirsa and Badoux [1990] are invaluable in considering these questions, with several viable possibilities available for any given structure -- (a) strengthening and embrittling, (b) strengthening and toughening, and (c) weakening and toughening. The latter solution, involving the deliberate weakening of a structure to increase its ductility, probably seems very much "off-the-wall" the first time it is seen by the typical structural engineer accustomed to designing new structures where increased strength is always viewed as desirable.

5. What are the appropriate seismic load levels for any given structure? Who should decide on the characteristics of the retrofit design earthquake? Does the engineer doing R/R have a special responsibility to get into such intricacies as the frequency content, and the issue of the effects of a "sharp jolt" type loading vs. a longer, more periodic motion?

6. How far should we go in insisting on the development of a rational, consistent retrofit strategy? A number of approaches have been proposed; some are based directly on interstory drift limits and others on energy considerations and associated damage levels at certain displacement levels. As an example, one attractive energy-based approach is that proposed by Rodriquez [1994] in which a parameter ID is defined for measuring the capacity of a given earthquake ground motion to damage structures. The value of ID is calculated from several quantities, including roof displacement at time t for the multi-story building subjected to the given ground motion, displacement at time t for a SDOF system subjected to the same ground motion, building height, building fundamental period, total hysteretic energy per unit mass dissipated by a SDOF system during the earthquake, and the maximum value of roof drift ratio associated with acceptable building performance.

   The ID parameter is then evaluated for each of several different retrofitting strategies, normally using two different design earthquake ground motions -- major, in which life safety may be the overriding issue, and moderate, in which building functional performance is also important. Rodriquez summarizes the process in a 9-step Redesign Procedure. Critical assumptions and estimates, such as values of roof drift ratio associated with acceptable levels of damage, and the global displacement ductility ratio of the retrofitted building, must be made by the engineer.

7. In evaluating the retrofitted structure (as for the original existing structure) what analysis approaches are appropriate? How will the level of analysis for R/R projects change over the next decade, and what effects will these changes have on R/R methodologies?

8. What is the effect of the "local element" (such as a new infill panel or shear wall) on the forces to be resisted by other components of the building, such as floor and

roof diaphragms, foundations, etc.? In other words, what is the local and global behavior of the total structural system action after retrofit?

9. What is the expected/intended/desired level of accuracy needed in R/R calculation approaches? How does this compare with accuracy levels commonly assumed in design of new buildings?

10. How many different types of retrofit should we try to fully develop? Can any one method be uniformly acceptable for low-rise, medium-rise, and high-rise buildings? How do we best quantify differences in requirements for high seismic areas as contrasted with those for moderate seismic areas?

11. What defines successful retrofit? Is limitation of interstory drift the single most important criterion, as proposed by Pincheira [1993]? How do we best educate the owner about the considerable differences between retrofit for life safety only vs. retrofit to guarantee post-earthquake operability? Some owners are content with ensuring life-safety only, while others may want to also protect building equipment and operations (Murray and Parker [1994]). In the latter situation, the permissible level of displacements and interstory drift values are sharply lower than for retrofit strategies addressing only life safety.

12. How important is it to avoid foundation overloading? A Cornell survey (Rao [1995]) of several practicing engineers on alternate R/R approaches indicates that possible overloading of foundations is an important issue, particularly since most strategies to strengthen foundations are very costly.

13. What's needed to enable the R/R process to be done with proper accounting for risk and reliability issues and life-cycle costs? How is risk defined (in the eyes of the building owner)? How do we best account for uncertainties in loading and in capacity? How do we best account for potential consequences (loss of life, cost) of failure?

14. How much does all of this cost? How we arrive at costs, realizing that they depend on such diverse factors as: (a) performance objectives for the retrofitted building, (b) site seismicity, (c) size and age of existing LRC building, (d) construction constraints as dictated by degree of normal building operation during retrofit, (e) lost building occupancy, and all the other local intangibles affecting final cost of construction.

3.2 Example -- Retrofit Decision Process for Critical Business Facility

Given these many issues and questions, it is not surprising that R/R engineering continues to be done so many times on an Ad hoc basis. As an example of how many of the above questions and issues have been dealt with in actual retrofit projects, the paper by Gates, Nester, and

Whitby [1992] will be mentioned here. They present a very interesting case study of the retrofit design process for an 8-story LRC frame built in Seal Beach CA in the mid 1960s under the provisions of the UBC-1964. The building houses the world headquarters for the Rockwell Corporation, which means that this case study is an excellent example of how to undertake decision-making for seismic retrofit of a critical business facility.

Design criteria were to assure life safety and minimize operational downtime after a damaging earthquake. After extensive studies, four alternatives were identified as candidates for building retrofit:

a. Base isolation plus exterior diagonal bracing
b. Conventional braced frames added to the exterior of the building
c. Exterior shearwalls in the perimeter frames
d. Jacketing of non-ductile beams and columns.

These four retrofit designs were carried forward and analyzed for relative risk, cost, etc., in comparison to the "do nothing" alternative. The building owner established a ranked listing of factors to be considered in reaching a decision on the best scheme, as follows: (1) life safety and minimization of bodily injury, (2) business interruption (vital administrative operations), (3) technical uncertainties associated with each strengthening concept, (4) disruption of vital building functions during construction, (5) comparative dollar losses resulting from the earthquake, (6) total project construction costs, (7) economic payback in terms of protection provided vs. total cost, and (8) aesthetic impact of the adopted scheme.

It is interesting to note that in order to help make the decision process more meaningful, the owner's management team was educated on several key earthquake engineering concepts, including the probabilistic approach to hazard prediction, definition of those conditions which typically result in closing of facilities by building officials after earthquakes, and areas of engineering uncertainty (ground motion, design and analysis, and constructibility) in predicting seismic structural behavior of the retrofitted building.

A comparison of the four retrofit schemes is summarized in Table 2, and risk assessment in terms of quantifiable costs for losses and retrofit is given in Fig. 1 (both are taken from the Gates, et al paper). Other non-quantifiable issues taken into account in the final decision process included life loss and bodily injury costs, present and future lost business opportunities, impact on the image and reputation of the company, and loss of confidence in building safety by its occupants.

The selected retrofit method was base isolation combined with strengthening of weak perimeter frames to ensure that they could safely resist expected inelastic deformations which might occur even with base isolators in place.

4.0 REVIEW OF SELECTED METHODS OF REHABILITATION/RETROFIT, EMPHASIZING THE USE OF CONCRETE AND/OR MASONRY

An excellent summary of the literature on R/R and repair of RC buildings is provided by Rodriquez and Park [1991], in which they describe R/R practices in the major seismic areas of the world, including Japan, the Balkan region, Mexico, and the U.S. Emphasis in the paper is placed on the repair and strengthening of reinforced concrete columns. The authors quote the statistics of Sugano [1981] and Endo et al [1984] on the frequency of use of various R/R and repair methods in Japan. Fig. 2 summarizes typical strengthening methods used in Japan, while Fig. 3 summarizes the methods actually employed for 157 buildings in Japan. Adding shear walls was the most widely used method by a substantial margin over reinforcement of columns, and the addition of wing walls.

The general effects of various concrete frame retrofitting schemes are summarized in Fig. 4, illustrating the gradually increasing stiffness and decreasing peak lateral displacement as a bare reinforced concrete frame is retrofitted with seven different schemes -- wingwalls on columns, precast panels, K-bracing, concrete block infill wall, diagonal braces, reinforced concrete infill wall, and integral wall.

Phan, Todd, and Lew [1993] specifically address strengthening methodologies for LRC frames, pointing out that seismic performance can be enhanced by increasing lateral strength, improving ductility, or using some combination of strength and ductility enhancement. They suggest seven possible schemes for improving LRC frame performance: (a) adding infill walls or steel cross-bracing, (b) adding new continuous shear walls or braces into existing frame geometry, (c) adding buttresses or frames on the frame exterior, (d) increasing thickness of existing walls or adding infill walls, (e) adding wing walls to columns, (f) jacketing existing columns and/or beams, and (g) strengthening existing joints.

4.1 Frame modification

Frame modification, designed to upgrade strength and/or stiffness and/or ductility, is the first category of R/R methods involving concrete construction. This approach is feasible if the number of separate elements needing modification is not excessive and the modified frame has sufficient stiffness to meet limitations on drift set by the R/R design criteria. Frame modification methods include: column (and beam) jacketing, combined column jacketing and post-tensioning, and joint region upgrading. Each of these methods are described below.

4.1.1 Column Jacketing

Column jacketing with additional concrete and longitudinal reinforcement running through the floor slabs is an effective and popular retrofit method. When the majority of columns are so strengthened, the additional lateral load capacity of the structure is uniformly distributed with minimal increased loading on any single foundation. The basic geometry of the building is not altered. However, this method does involve extensive construction activities and consequent disruption of the building occupancy. The second workshop paper by Jirsa provides a review of jacketing research done at the University of Texas.

4.1.2 Combined Column Jacketing and Column Post-Tensioning

Bracci, Reinhorn, and Mander [1992] (also reported in Reinhorn, Bracci, and Mander [1993]) at SUNY-Buffalo utilized a combination of column jacketing and post-tensioning to retrofit the interior columns of a three bay, three story 1/3 scale reinforced concrete building tested on the SUNY-Buffalo shake table. Column size was increased from 100 mm by 100 mm to 150 mm by 150 mm with the enlarged column reinforced with four high-strength threaded steel rods placed in sleeves (to prevent bond) from the roof level to the mid-height of the first floor. The thread-bars were bonded in the jacket concrete in the lower half of the first floor to properly anchor the post-tensioning force applied at the roof level. They were intentionally discontinued at the rigid base of the model building to avoid imposing any additional bending moment on the foundation.

The post-tensioning provided several benefits, including enhancement of the shear capacity of the column and the beam-column joint region, provision of an initial strain in the new composite section of the jacketed column to help ensure compatibility of the section, and provision of a compressive ("clamping") action on the discontinuous bottom level beam reinforcement to delay possible pullout.

The retrofit strategy is illustrated in Fig. 5, where it may be seen that some additional reinforcement of beams framing into columns was accomplished by casting a reinforced concrete "fillet" at the top of each column. The retrofitted structure was subjected to two versions of Taft N21E -- 0.20PGA and 0.30PGA. Response of the retrofitted structure was judged to be satisfactory, with critical damage moved away from the previously weak columns. During the higher level table motion, an incipient beam-sidesway mechanism was under development.

Bracci et al designed two other similar retrofit schemes -- a prestressed masonry block jacketing (Fig. 6), and prestressed partial masonry infills (Fig. 7) also termed wingwalls by some engineers. The masonry block jacketing should accomplish the same results as with additional concrete around the column. A wire mesh in the mortar bed joints could be used to improve shear capacity of the new composite column and enhance continuity of the masonry jacket with the existing column. As with the concrete jacket, the use of a reinforced concrete fillet at the joint area is recommended. The partial masonry infill provides some of the benefits of a wall without closing off the space between columns. The partial wall would extend no more than several blocks from the column; it could be either symmetrical or unsymmetrical at interior columns. Vertical reinforcement in these masonry units should extend through the floor slab, and transverse reinforcement and masonry bed joint reinforcement should also be used.

4.1.3 Upgrading of Joint Region

Youssef and Hilmy [1990] describe the use of additional reinforcement to provide positive moment capacity and plastic rotation capacity in beams with discontinuous bottom steel, as shown in Fig. 8. This method was used in retrofitting a 12-story office building in downtown Long Beach, a 1970s building with bottom reinforcement lap spliced within the column. Anchoring the new reinforcement (which passed through the column) to the existing beams was done with multiple epoxy grouted anchors.

The same approach was studied by Beres, El-Borgi, White, and Gergely [1992] for use at both interior and exterior columns of LRC frames. Reversed cyclic loading experiments were conducted on full-scale joint regions retrofitted in this fashion. Interior and exterior joint retrofit geometries are illustrated in Figs. 9 and 10, respectively. Interior joint retrofit used exterior bars anchored to channel sections bolted to the beams on each side of the column, while exterior joint retrofit utilized high-strength bolts and steel plates to provide positive moment capacity and also to suppress the undesirable failure mode of spalling of concrete off the back of the joint and subsequent debonding of the bent-down negative moment reinforcement.

The retrofitted interior joint performed well; pullout of the discontinuous bottom reinforcement was prevented and damage was transferred from the embedment zone to other parts of the joint region. Column shear strength was enhanced and the deterioration rate of the joint region under cyclic loadings was reduced. Stiffness characteristics and energy dissipation showed no significant changes from a non-retrofitted companion specimen subjected to the same cyclic loading history.

The retrofitted exterior joint had major changes in behavior as compared with a non-retrofitted companion specimen. Beneficial changes included the formation of a flexural hinge in the joint panel zone close to the beam, complete protection of the back concrete cover, and prevention of joint region cracks from extending into the top column splice region. Stiffness was increased only slightly and peak strength was increased substantially because of the plastic bending action of the back plate at high deformation levels.

While this approach can provide only limited increased capacity and ductility, it is unobtrusive, easy to implement, adds little stiffness to the frame, and may permit strengthening of exterior joints in some buildings without having to break the exterior facade envelope.

4.2 Supplementary lateral resistance and stiffness

Supplementary lateral resistance and stiffness is normally required when the conditions for economical frame modification (as stated above) cannot be met, or the retrofitted building must have post-earthquake operational capabilities. Several approaches may be considered:

(a) Providing totally separate (new) cast-in-place shear walls, with vertical continuity of reinforcement,
(b) Using infill walls placed within selected frames in the building,
(c) Adding new ductile frames to act as back-up systems for the LRC frame.

4.2.1 Cast-in-Place (CIP) Shear Walls

The addition of new shear walls has been used in retrofitting non-ductile RC frame buildings, particularly when the retrofit design criteria calls for controlling displacements to levels that can guarantee post-earthquake operability. Murray and Parker [1994] report on the use of this approach in retrofitting several Central U.S. buildings ranging from 1 to 3 stories with floor areas of 25,000 to 62,000 square feet. Typical framing was concrete flat slab or flat plate with spandrel beams, and with exterior walls consisting of unreinforced masonry infill set tightly within the concrete framing (columns and spandrel beams). The masonry was multi-wythe brick or brick veneer with hollow concrete block backup.

It was decided to retrofit with RC shearwalls to provide the stiffness needed to limit lateral displacements and resulting damage to the existing unreinforced masonry walls and non-structural components. The Southern Building Code was used with effective peak ground accelerations ranging from 0.14g to 0.24g. Probabilistic estimates were based on a 475-year return period, while the deterministic design criteria were based on a 7.6 magnitude earthquake located within the New Madrid Seismic Zone. Murray and Parker [1994] provide considerable detail on the design base shear and the distribution of forces over the height of the structures.

The new shearwalls were normally placed adjacent to existing exterior masonry walls, with some interior walls needed to reduce diaphragm spans or because of partial upper floors. The boundary forces induced in columns at the ends of new shearwalls from overturning action were accounted for, as were effects of the walls on foundation capacities. Shearwalls ranged in thickness from 150 to 250 mm, with thickness normally controlled by shear (not flexure). In some cases wall thickness and/or concrete strength were increased to limit the factored shear force to the level which permitted building the wall with only a single mat of reinforcement. Single mats were used whenever possible because of the tight and difficult working conditions.

A typical detail at an exterior wall is shown in Fig. 11. Vertical reinforcing was placed through 2-inch diameter holes core-drilled in the slab. Horizontal reinforcing was lap-spliced with dowels epoxied into the columns as shown in Fig. 12. Wall-column connection integrity was further enhanced with a bonding agent applied to the face of the column.

Other critical design issues involved in this retrofitting operation included: (a) determining the adequacy of existing floor and roof slabs to carry the seismic forces, (b) transfer of diaphragm shears into the new shearwalls with vertical dowels, (c) adding new collector and drag members to the diaphragms, (d) providing new tubular steel section strongbacks to both exterior masonry infill walls and interior masonry partitions to provide out-of-plane strengthening and stiffening to limit out-of-plane deflections to h/400, (e) anchorage and bracing of masonry parapets and stone ornamentation, and (f) anchoring existing brick veneer to the backup masonry units with a remedial wall tie system.

Typical "upper range" unit costs (including contractor's fees and general conditions) for this type of post-earthquake operability retrofit of low-rise LRC frames in the central U.S. are given in Table 3. Buildings were occupied and fully functional during the retrofit operation.

The reactions of the new shear walls on existing foundations may cause serious problems and are a potential strong disadvantage of this R/R method. Another disadvantage is the closing of formerly open spaces, which can have major negative impact on interior building uses or exterior appearance.

Pincheira [1993] provides detailed comparisons of performance of a 3-story RC frame retrofitted with a structural wall, with post-tensioned bracing, and with X-bracing. The strengthening schemes used in this study are shown in Fig. 13 and comparative analytical results of performance of the three retrofit schemes are given in Fig. 14. Note that the wall scheme has very high initial stiffness, pointing out the potential strong disadvantage of adding excessive retrofit stiffness and potentially producing a sharp increase in seismic forces.

4.2.2 Infill Walls -- Masonry

There is an extensive international literature dating back to the 1950s and 1960s on experiments and analysis of infilled frames, and only very selected samples of more recent research will be reviewed here.

Harris, Ballouz, and Kopatz [1993] studied the performance of 3-story, single bay LRC frames retrofitted with block masonry infill to resist moderate earthquakes. Their literature review pointed out the high sensitivity of frame performance to relative values of infill strength, column stiffness and strength, and beam strength. They describe the three basic failure modes for masonry infilled frames as quantified by Liauw and Kwan in earlier papers:

a. Mode 1 with weak columns, strong beams, and strong infill -- failure occurs in the column, followed by crushing of infill in the compressive corners (Fig. 15a).
b. Mode 2 with strong columns, weak beams, and strong infill -- failure occurs in the beam, again followed by corner crushing of infill (Fig. 15b).
c. Mode 3, with strong frame and weak infill -- failure occurs when corner crushing extends through the diagonal, followed by frame joint failure Fig. 15c).

In the Harris et al study, four 1/6 scale frames infilled with masonry were subjected to in-plane monotonic triangular shear loading. One was tested with no infills, one with infill on the first story only, one with infills in stories 1 and 2, and one with infills in all three stories. The three infilled frames and associated failure modes are shown in Fig. 16. The three story infilled frame (the first infilled frame tested) experienced significant damage only in the first floor infill, and it was decided to test the remaining two infilled frames with metal braces in the unfilled floors so that the same triangular load distribution could be used for all frames. Details of the testing program and results are given by Ballouz [1994].

Both strength and stiffness values of the infilled frames were about an order of magnitude higher than in the bare frame. In each frame the first floor infill eventually cracked along a horizontal mortar joint, splitting the infill into two parts which then greatly increased column bending action. Failure resulted from yielding of the first floor tension column in the undesirable Mode 1, as shown in Fig. 17.

Conclusions reached in this study include:

1. Retrofitting with non-integral masonry infills is an effective and economical method for improving strength and reducing drift of LRC frames.
2. Substantial strengthening of the LRC frame was achieved, but the relatively strong masonry infill used in these tests resulted in a catastrophic failure of the column.
3. By proper selection of the infill masonry strength, along with prevention of its premature separation from the columns, a more desirable failure mode can be achieved, with yield hinges at the top and bottom of the columns and crushing of the masonry infill.
4. Anchorage of the masonry to the frame is a critical factor in determining overall performance. With proper anchorage, it should be possible to force failure in the masonry and prevent a premature shear/flexure column failure.

Zarnic [1994] and Zarnic and Tomazevic [1984,1985] present considerable experimental results on cyclic load tests on infilled frames. They quantify the substantial strength and stiffness increases provided by properly detailed and constructed concrete masonry infills. They use these (and other) test results to explain the dramatic changes in behavior of the frame-infill combination as lateral loading increases to the level of masonry cracking. Prior to cracking, strain levels are low and the frame-infill behaves as a monolithic cantilever shear wall. After significant infill cracking, separations of the infill occur both within the wall itself and along its boundaries, and the distribution of lateral load to the bounding frame becomes highly non-uniform. The frame members are called on to resist much higher forces, and the columns in particular may become "captive columns" with concentrated shear loads away from the joints, overloaded to the point of incipient collapse. The infill is still quite capable of carrying high loads even when it is severely cracked, provided the frame can supply adequate constraining action.

They use these physical arguments to propose that the infill should be modeled analytically on the basis of the panel action of the infill rather than with an equivalent strut. The panel action model is better suited to capture the real post-cracking behavior of the masonry.

4.2.3 Infill Walls -- CIP Reinforced Concrete

Altin, Ersoy, and Tankut [1990] provide extensive experimental data on seismic strengthening of RC frames with CIP RC infill walls. Fourteen two-story, one-bay infilled frames (infill panel dimensions of 750 by 1300 by 50 mm) were tested under reversed cyclic load to simulate seismic effects. Four different types of infill reinforcement and connection details were included in the tests. The effects of column axial loads and column flexural capacity on frame behavior and strength were also studied. Test results are presented in terms of effect of infills on stiffness, strength, ductility, energy dissipation, and lateral drift. Measured results are compared with calculated strength and stiffness values using empirical equations.

Lateral strengths of the infilled frames connected to frame members ranged from about 200% to 600% greater than bare frame strengths, while the unconnected infill provided an increase of about 140%.

Initial stiffness of infilled frames was at least 11 times higher than that of bare frames. Projecting the results to prototype 3-story buildings indicated that the building period would decrease by about 80% if all frames were infilled and by about 70% when 40% of the frames were infilled. Period changes were primarily due to stiffness changes and had only slight dependence on the increased mass of the infills.

This research study showed quite clearly that the pattern of reinforcement used in the infill walls had little effect on peak lateral strength. The most significant parameter in defining lateral strength was the type of connection between infill and frame members. Reversed cyclic loading accentuates the importance of connections. The best performance is obtained by setting dowels into the frame members and then lapping infill reinforcement with the dowels.

The authors recommend that infilled frame lateral strength should not be taken higher than 80% of that of monolithically cast frame/wall systems. They also point out that selective column strengthening prior to placing infills may improve overall performance by delaying brittle failure mechanisms.

4.2.4 Infill Walls -- Precast Concrete Panels

The use of modular precast panels is an attractive option for retrofitting LRC frames, with minimal on-site concreting operations and panel units capable of being delivered in building elevators and handled with modest equipment. Connections between panels and frame are critical. M. Kreger et al [1995] have done recent research on a system that combines precast reinforced concrete infill walls and post-tensioning of boundary members to strengthen and repair nonductile RC frames. The concept is to utilize the shear capacity of the infill walls along with the moment capacity created by the post-tensioning tendons in the frame boundary members.

Precast panel weight was set at 8.9 kN (2 k) maximum to permit moving the panels into buildings on elevators. With a 152 mm (6 in.) thickness, the panel size is thus limited to about 1.52 m by 1.52 m (5 ft. by 5 ft.), with multiple panels needed for a typical frame opening. Panels are cast with teeth-like shear keys around all four edges, and are connected together with closure strips made from 1.06 MPa (7.3 ksi) strength concrete. The clear distance between adjoining panels is set at 51 mm (2 in.), with 2-#3 reinforcing bars placed in both the horizontal and vertical closure strips. Panel to frame connections were made by coring holes into the bounding beam and column members and embedding 76 mm (3 in.) diameter steel pipe dowels which in turn were fastened into the panel and closure strip. The shear failure capacity of this type of precast wall is governed by the shear strength of the steel pipe dowels, thus producing a ductile failure mode.

A series of 15 multiple panel tests have been completed and a 2/3 scale two-story, one-bay nonductile RC frame specimen using the precast infill walls and post-tensioning of boundary members in under study.

4.2.5 Partial Infill Walls

The use of partial walls may be considered as an intermediate solution between frame modification and providing additional lateral resistance. This approach provides enhanced strength but with substantially smaller stiffness increase than that associated with full infills. It also does not require complete filling of frame openings, thus giving the method an important functional advantage over full infills.

Partial walls (piers, wing walls) built adjacent to existing columns may be either reinforced concrete or masonry. Roach and Jirsa [1986] retrofitted a 2/3 scale model of a two-story, two-bay frame (consisting of deep spandrel beams and short slender columns) using 1524 mm (60 in.) wide reinforced concrete piers designed to increase the column capacities and force the failure mode into flexural hinging in the beams. The conclusions from this study are as follows: "(1) The RC piers acted as strong columns which developed the flexural capacity of the beams and caused frame to fail in a ductile manner, (2) The pier-strengthened frame had an initial stiffness three times the initial lateral stiffness of the existing frame, (3) The reinforced concrete piers increased the lateral strength to at least five times the calculated lateral strength of the existing frame, (4) The loads applied on the frame did not develop the shear capacity of the piers and developed a small percentage of the flexural capacity of the piers, and (5) The adhesive bond and dowel action provided adequate load transfer between the reinforced concrete piers and the original frame."

The proposals of Bracci et al [1992] have been discussed briefly in Section 4.1.2. The effectiveness of partial walls are a strong function of relative wall proportions and particularly of wall-to-frame connection details.

5.0 RESEARCH NEEDS AND DIRECTIONS

A detailed listing of research needs in this first paper of the Workshop might be counter productive in that it could provide too much input for the actual workshop sessions where participants will be brainstorming these very issues. Thus the approach taken here is to provide a few general comments and then discuss a listing of research needs for two retrofit systems -- CIP reinforced concrete panel infills, and masonry infills.

5.1 General Issues In Defining Research Needs

Several thoughts come to mind in trying to decide on strategies for additional research and development of retrofit methodologies. These include (in no particular order of priority):

1. A catalog of feasible retrofit methods is needed by the design profession because of the great variability of retrofit requirements from one building to the next.
2. In a typical retrofit, only selected portions or components of the structure are modified or enhanced, but in an earthquake, all components on each floor, retrofitted or not, will undergo essentially the same lateral displacements. Thus the ability to predict initial and degraded stiffness of the retrofitted structure, and hence the expected drift characteristics, is critical.
3. Expected failure modes of the actual retrofitted structure need clarification and quantification.
4. Retrofit schemes that provide only relatively limited benefits may be all that is needed for certain classes of structures in moderate seismic zones. Thus methods that are not deemed applicable for high seismic zones may well be promising candidates for further study and application.
5. Feasibility and practicality of construction must be given very high priority in deciding how to allocate resources for retrofit research.
6. Costs of a given retrofit scheme should not be separated from the more technical considerations. Economic comparisons must include cost of the retrofitting operations themselves plus any "downtime" costs for the building occupants

5.2 Research Needs -- Frames Infilled with Reinforced Concrete Panels

Altin, Ersoy, and Tankut [1990] outline a number of research needs for frames with infill panels:

1. Define optimum thickness of the infill panel considering strength, stiffness, and energy dissipation characteristics.
2. Study the effect of differing levels of axial force.
3. Consider strengthening frame columns at the base of the structure, prior to infilling the frame, to delay or prevent the brittle shear-sliding failure modes observed in all specimens.
4. investigate the effectiveness of partial infills and infills with openings.
5. Determine the optimum amount and location of infill panel reinforcement.
6. Test multi-bay frames to determine if there are differences in behavior and strength as implied by the single bay experiments.

5.3 Research Needs -- Frames Infilled with Masonry

Abrams [1994] provides a four page summary of resolutions pertaining to masonry infills which resulted from the NCEER international workshop on seismic response of masonry infills.

Workshop participants agreed that a compressive strut model can reasonably represent the in-plane panel stiffness, and that the properties of the strut may be developed with the use of physical or numerical models, or from semi-empirical expressions. An abbreviated listing of research needs includes the following:

1. Establishment of global drift limits for frame-infill systems to assure that local panel performance criteria are met.
2. Incorporation of the effect of vertical loads into equivalent strut models.
3. Introduction of biaxial material properties for modeling infill behavior near the corners of panels.
4. Establishment of appropriate levels of global damping.
5. Better understanding of the behavior of panels with openings.
6. Determination of criteria for formulation of equivalent strut models in terms of system drift and local panel deformation and degradation.
7. Consideration of the feasibility of simple methods for estimating the seismic strength of infill panels such as a nominal average shear stress or plastic analysis methods or equivalent strut models.
8. Establish behavior of masonry infills under bi-directional ground motions, particularly out-of-plane stability under large in-plane deformations.
9. Explore the feasibility of developing performance-based design methods that rely on knowledge of stiffness and damage at various levels.
10. Study behavior of infills in weak, non-ductile frames.

6.0 SUITABLE LOAD HISTORY PROTOCOLS FOR LABORATORY TESTS

Comparison of experimental research results on retrofit schemes is complicated by the lack of agreement on suitable loading (deformation) histories to be used in conducting the tests. Two basic types of test protocols are needed for element tests -- one for beam-column connection regions, where biaxial bending may be utilized, and one for essentially in-plane tests on wall elements and wall-frame combinations. More complex protocols may be needed for testing sub-assemblages or complete structures. In any case, testing is almost always done on a deformation-controlled basis to best facilitate interpretation of results in terms of ductility and also to permit continued testing when load capacity is decreasing.

Pertinent variables include the level of loading (deformation) in early cycles, the load (deformation) increment from one set of cycles to the next, and the sequencing of cycles when biaxial load histories are used. Typical protocols begin at drift levels of 1/4% to 1/2%, with either 2 or 3 cycles at this initial deformation level. Increases in peak drift of either 1/4% or 1/2% are used for each set of 2 or 3 cycles, continuing to failure. Some investigators apply a low level cycle in between each set of cycles. A decision on whether to use 2 or 3 cycles at a given deformation level can be made on the basis of change between the first and second cycle -- if there is no significant change, then 2 cycles is adequate; if there is a significant change, then 3 cycles should be applied at a given deformation level.

6.1 The ATC Recommendations for Cyclic Testing of Steel Structures

The specification of loading histories has been studied extensively for cyclic load testing of steel structural elements and configurations, as reported in ATC [1992]. The ATC recommendations for loading history are divided into two categories: Single Specimen Testing Program and Multi-Specimen Testing Program.

The single specimen program is used when (a) only one specimen is available, (b) the monotonic load-deformation response (say up to at least the yield point) can be predicted with confidence, (c) strength deterioration occurs slowly, and (d) analytical cumulative damage modeling is not being attempted. The recommended testing program, shown in Fig. 18, consists of stepwise increasing deformation cycles (the so-called multiple step test). The ATC document contains detailed recommendations for numbers of cycles at a given deformation, when to use more cycles per step, etc. The procedure also recommends that it may be advisable to interrupt the loading history with small cycles (typically at deformations corresponding to 3/4 of the yield deformation) to evaluate intermittent stiffness degradation.

The multi-specimen testing program is recommended when (a) the monotonic behavior cannot be predicted with reasonable confidence, (b) onset of strength deterioration shows large scatter, (c) strength deterioration is rapid, or (d) a cumulative damage model needs to be developed for assessing seismic performance under arbitrary loading histories. Additional details on multi-specimen testing are given in the ATC report.

It is suggested that the Retrofit Workshop participants (perhaps only a subset of the entire group) give some limited attention to possible endorsement (or modification) of the ATC guidelines for use in testing retrofit specimens and configurations.

6.2 Column/Connection Region Specimens

Alcocer and Jirsa [1990] utilized a bidirectional cyclic load history in testing four full-scale interior frame connections. This loading protocol, shown in Fig. 19, is designed to represent a severe loading condition for a joint region, with applied interstory drift angles of 0.5%, 1%, 2%, and 4%. This same loading scheme was used earlier in the US-Japan-New Zealand Cooperative Research Program on joint design and in another test program by Guimaraes at U. Texas to study the effect of high performance materials on joint behavior.

6.3 Dynamic Testing Load Histories

Two types of tests are done on structural subassemblages or on complete structures to predict true dynamic response:

1. The first being shake table tests with actual earthquake ground motions programmed into the shake table (earthquake simulator), typically starting out with a scaled-down ground

motion and then gradually increasing the intensity until the maximum desired motion has been applied to the structure. A question to be faced in this method of testing is the level of the first ground motion, which can depend on the sensitivity of the structure to the total amount of energy it receives before significant deterioration in stiffness or strength. Three or four successively increasing table motions are typically used in progressing to the final loading; for some structures a higher level loading is applied very early in the testing sequence to better simulate an undamaged structure being hit by a major earthquake. While shake table testing is certainly the closest representation of the real loading on a structure during an earthquake, with the structure generating spatially correct inertia loads according to its changing stiffness properties, the test is over so quickly that correlation of visible damage with load intensity is difficult if not impossible during a run, and one gets no "second chance" to get data if there is a malfunction of the electronics.

2. The second type is pseudo-dynamic testing where the structure is tested with imposed quasi-statically displacements that resemble those that would be developed had the structure been tested dynamically. This approach can be summarized as a six-step process: (1) the tested structure is idealized as a discrete-parameter system, (2) the equations of motion for the system are formulated as a set of second-order differential equations, (3) the inertial and viscous damping characteristics are numerically specified, (4) structural restoring forces are measured directly during the experiment and subsequently used in the calculations, (5) step-by-step numerical integration of the governing differential equations of motion for the tested specimen is performed using an on-line computer, and (6) the computed displacement response, corresponding to a specific earthquake excitation, is imposed on the tested structure by means of servo-hydraulic actuators controlled by the on-line computer.

The pseudo-dynamic approach produces a set of realistic seismic loads on the test structure, including the correct floor-by-floor forces for multi-story structures. It also has the advantage that one can see what is happening to the structure because the very short-time dynamic loading is lengthened out to a test that may take a day or even longer, thus permitting quantification of damage as loading progresses. As with the shake table testing approach, the usual procedure is to begin with a low level ground motion, and gradually ramp up with perhaps three or four increasingly severe motions until the maximum level of loading is achieved, or the structure fails. If failure is not reached, the structure can be loaded unidirectionally to collapse after completion of the simulated dynamic testing.

7.0 RETROFIT GUIDELINES AND THEIR DEVELOPMENT

Suggestions for the format and level of detail of retrofit guidelines are an expected product of this workshop. As an initial suggestion, it is proposed that the guidelines be developed as a natural extension of the material presented in FEMA-172 [1992]. The guidelines should include, for each methodology included in the catalog of methods:

a. Typical ranges of structural performance characteristics (initial and degraded stiffness, strength, expected levels of ductility) that may be achieved,
b. Engineering design approaches for achieving these characteristics,
c. Detailed recommendations for connection of retrofit components to the frame,
d. Suggested simplified analytical models (or modelling assumptions) to be used in analyzing the response of the retrofitted structure, and in particular, in determining a reasonable value of expected drift values (an example might be recommendations such as the inelastic response spectra approach put forth by Pincheira [1993] for determining minimum requirements of stiffness and strength for adequate performance of the retrofit scheme),
e. Impact of retrofit on foundation loads,
f. Potential advantages and disadvantages for typical applications.

8.0 SUMMARY

There are many diverse issues pertaining to selection of retrofit strategies for existing reinforced concrete framed buildings designed primarily for gravity loads, and a high level of engineering expertise is required for successful retrofit design. Retrofit methods are reviewed for selected techniques based primarily on concrete and masonry components. Loading protocols to help unify experimental study of retrofit performance are presented. The paper concludes with suggestions on format and content of retrofit guidelines.

ACKNOWLEDGEMENTS

Results from previous research projects supported by NIST and NCEER are incorporated into this paper, and the support of these organizations is gratefully acknowledged. I also want to thank Gerry Cheok of NIST for her helpful comments on an early partial draft of the paper, and Peter Gergely for his excellent and continuous help and assistance during our many years of working together on seismic performance of concrete structures.

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Table 1. Comparison of Prototype Structures.

		12-Story Prototype		3-Story Prototype	
		WSD	**USD**	**WSD**	**USD**
Triangular Profile	Base shear	8.8%	7.9%	32.6%	24.3%
	Roof drift	1.32%	0.97%	0.34%	0.48%
	Mechanism	Floors 1-9	Floors 7-8	Floors 0-1	Floors 0-1
Rectangular Profile	Base shear	11.6%	-	34.7%	24.9%
	Roof drift	1.20%	-	0.28%	0.39%
	Mechanism	Floors 0-7	-	Floors 0-1	Floors 0-1

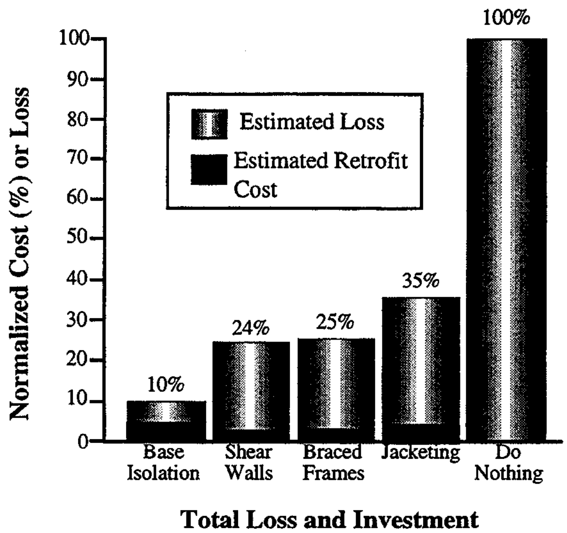
Table 2. Comparison of Alternative Retrofit Schemes.

Partial List of Seismic Risks @ MCE	#1 Base Isolation	#2 Braced Frames	#3 External Shearwalls	#4 Jacketing	Do Nothing
• Life Safety - Injury	Minor	Moderate	Moderate	Moderate	Extensive
• Life Loss	Not Expected	Not Expected	Not Expected	Not Expected	Some
• Equip. Damage	Minor	Moderate	Moderate	Moderate	Extensive
• Business Int.	Hours-Days	Weeks	Weeks	Weeks-Months	Months or Relocation
Construction					
• Business Impact	Low	Medium	Medium	High	--
• Architectural	Low-Mod.	Low-Mod.	High	Low	--
• Schedule (Yrs.)	3	1.75	2	1.5	--
• Project Cost	2.2	1.0	1.2	1.0	--
Impact of Eng. Uncertainties					
• Ground Motion	High	Medium	Medium	Low	--
• Design & Analysis	Low	Low	Low	Low	--
• Constructibility	Medium	Low	Low	Medium	--
History of Performance in Earthquakes	Some	Moderate	Extensive	Some	Extensive

Table 3. Unit Costs for Retrofit.

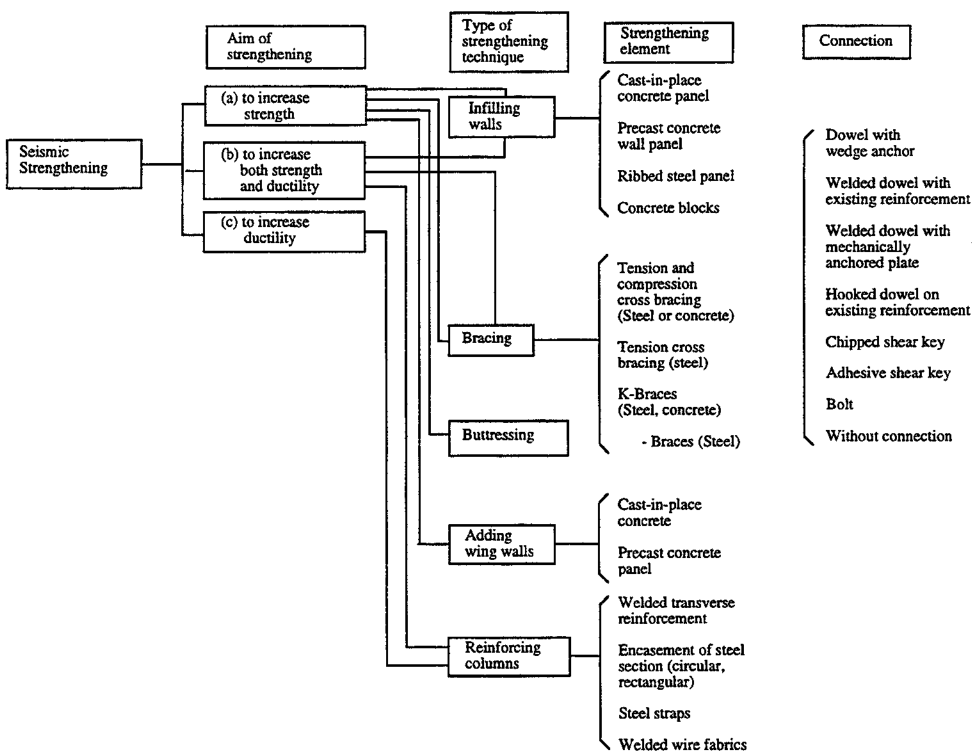
Shearwalls[1]	$40 - $55 / Sq. Ft. of Wall
Exterior Wall Strongbacks	$325 - $375 / Each
Interior Wall Strongbacks	$250 - $325 / Each
Parapet Strengthening[2]	$50 / Linear Ft.
Chord/Collector Angle or Plate	$55 - $80 / Linear Ft.
Brick Veneer Ties	$14 - $28 / Each
Bracing of Lights and Ceiling	$1 - $1.25 / Sq. Ft. of Ceiling Area
Bracing of Suspended MPE Components[3]	$3 - $5 / Sq. Ft. of Building Area
Bracing and Anchorage of MPE Equipment	$300 / Piece of Equipment

[1] Includes demolition of existing wall elements, as required, but does not include new wall finishes.
[2] Center Coring Technique [8]
[3] Includes relocation of these components, as required, during construction.



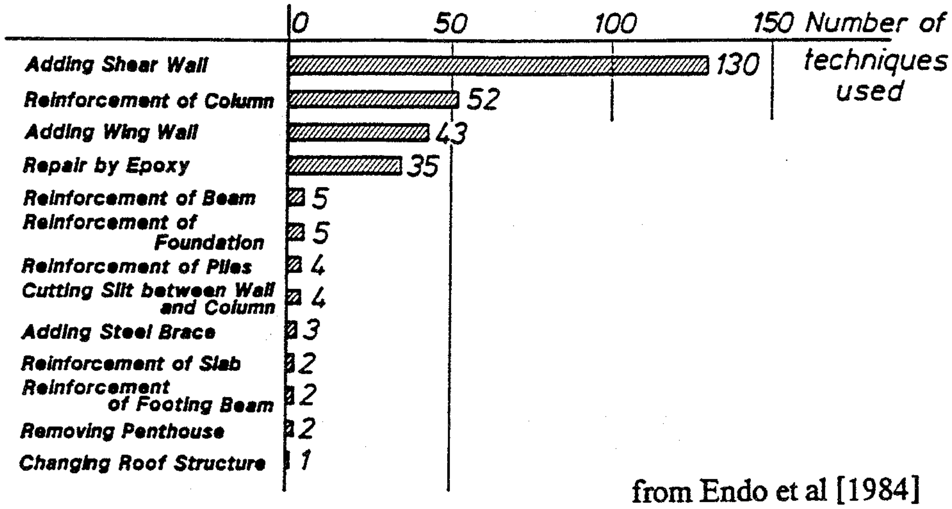
from Gates, Nester, and Whitby [1992]

Figure 1 Risk Assessment Summary in Terms of Quantifiable Costs



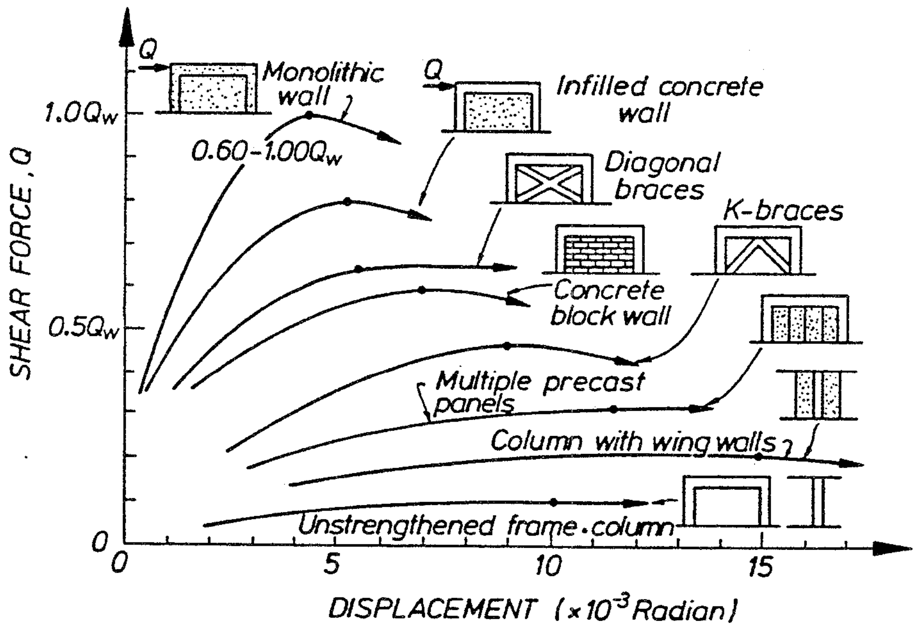
from Sugano [1981]

Figure 2 Typical Strengthening Methods Used in Japan



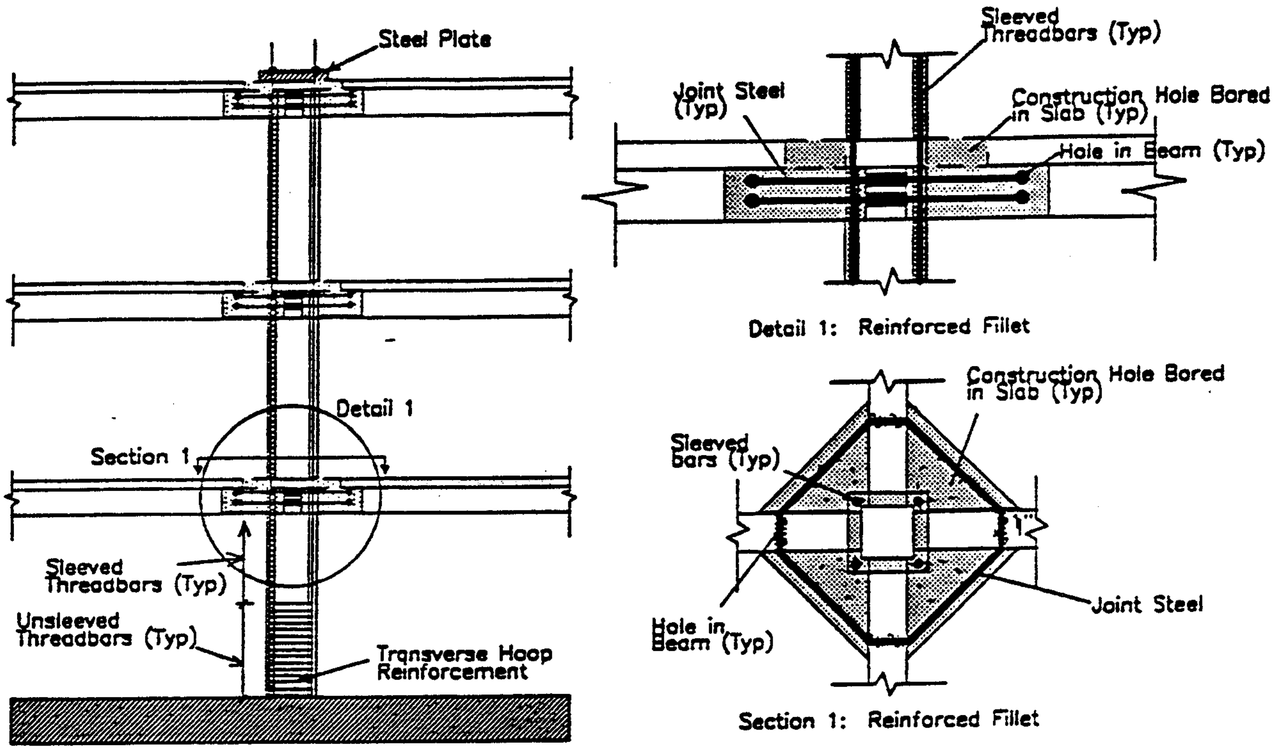
from Endo et al [1984]

Figure 3 Repair and Strengthening Techniques Used for 157 Buildings in Japan



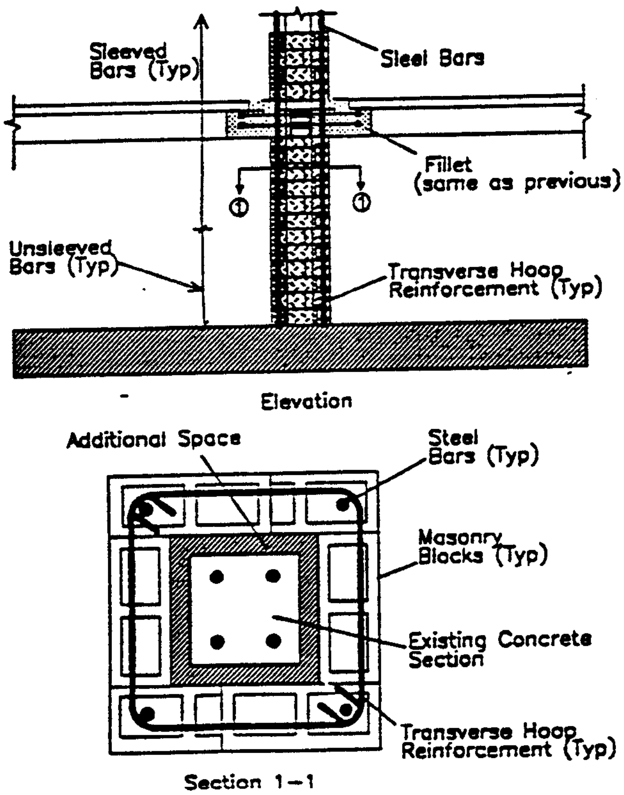
from Sugano [1981]

Figure 4 Typical Load-Displacement Relationships for Different Strengthening Techniques



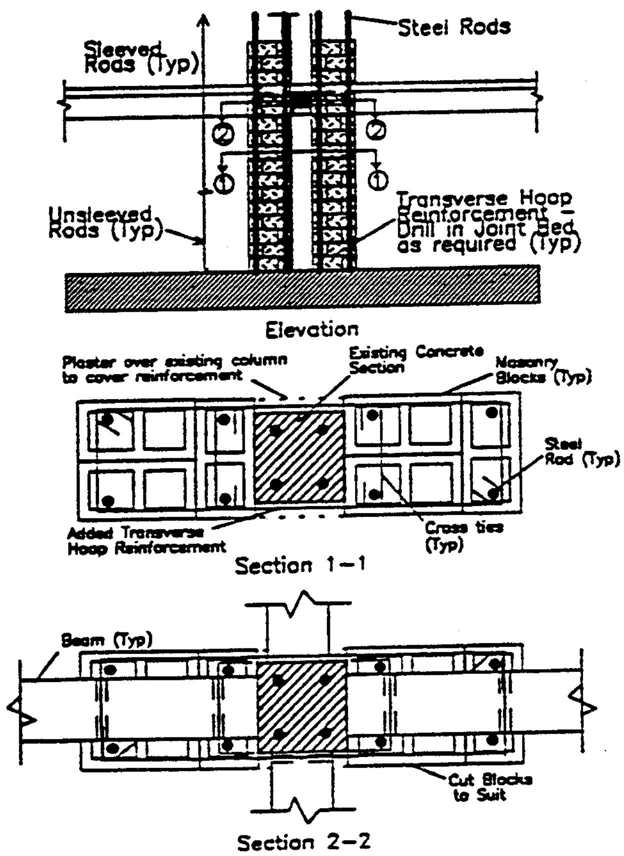
from Bracci, Reinhorn and Mander [1992]

Figure 5 Improved Concrete Jacketing Technique



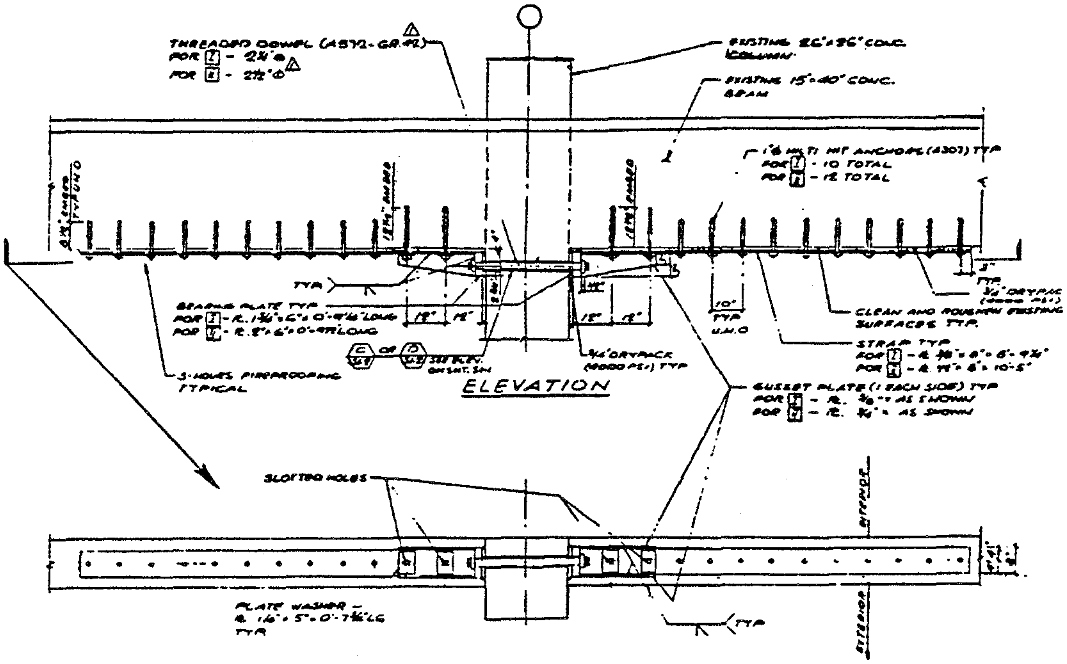
from Bracci, Reinhorn and Mander [1992]

Figure 6 Masonry Jacketing Technique



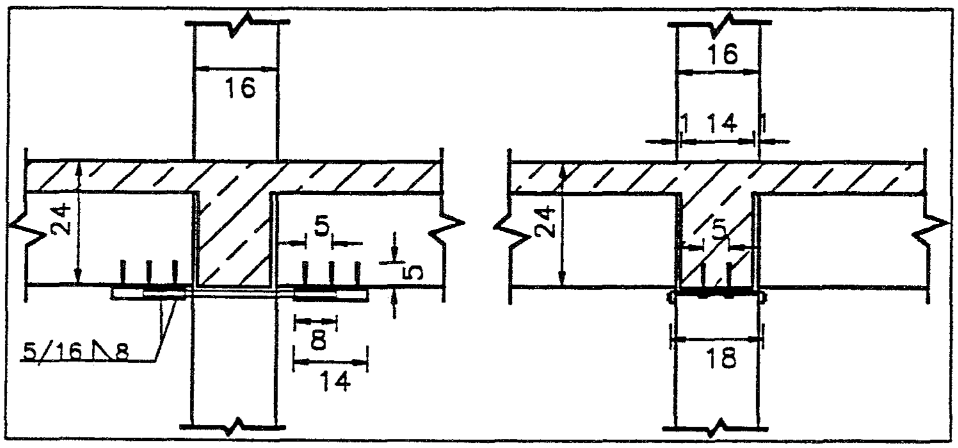
from Bracci, Reinhorn and Mander [1992]

Figure 7 Partial Masonry Infill Technique



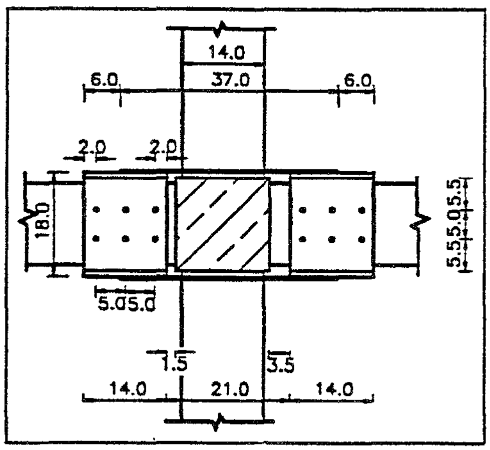
from Youssef and Hilmy [1990]

Figure 8 Strengthening Details of the Beam-Column Joint



a. Elevation View 1.

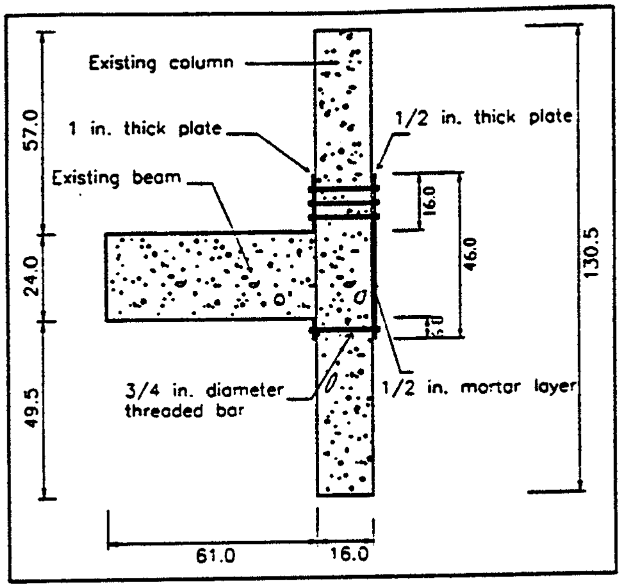
b. Elevation View 2



from Beres, El-Borgi, White and Gergely [1992a,b]

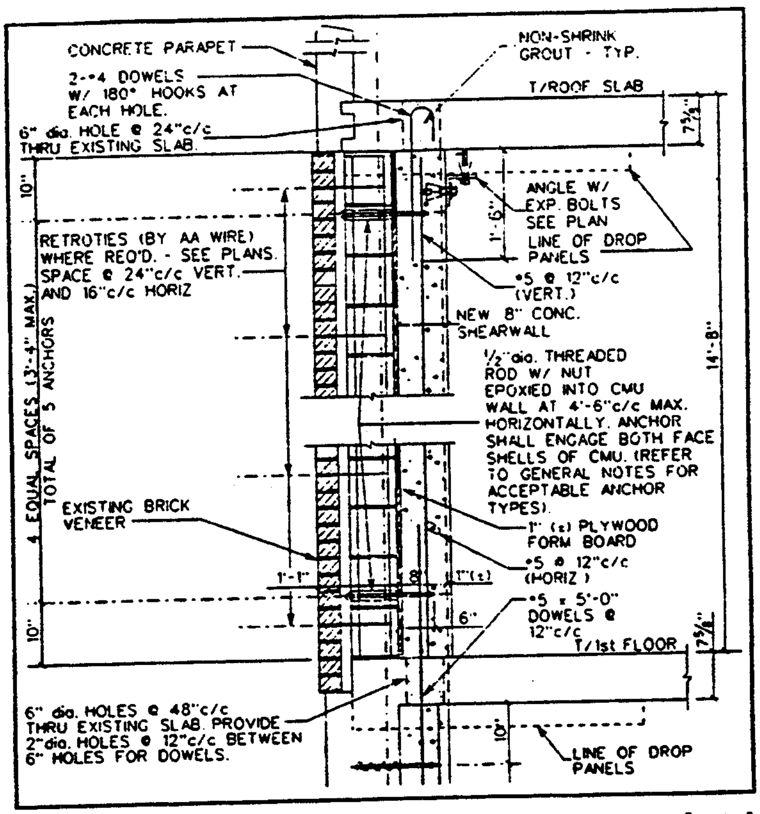
c. Bottom View

Figure 9 Interior Joint Retrofit Configuration



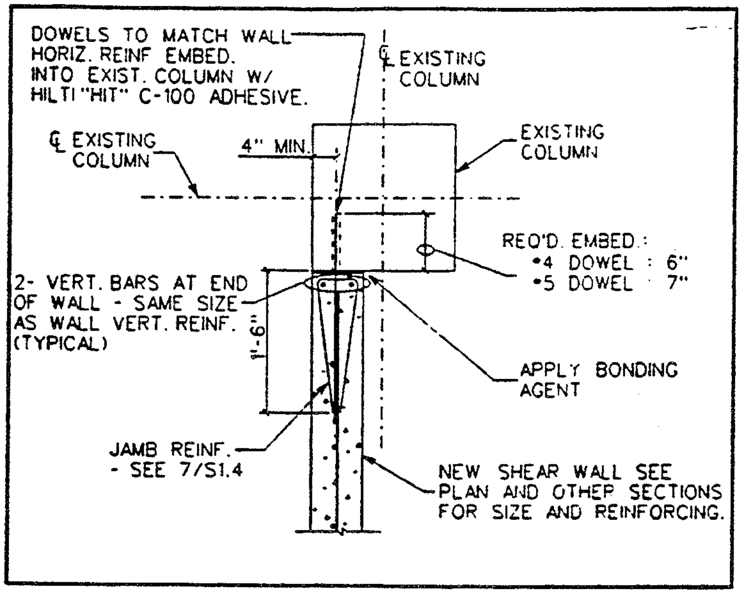
from Beres, El-Borgi, White and Gergely [1992a,b]

Figure 10 Exterior Joint Retrofit Configuration



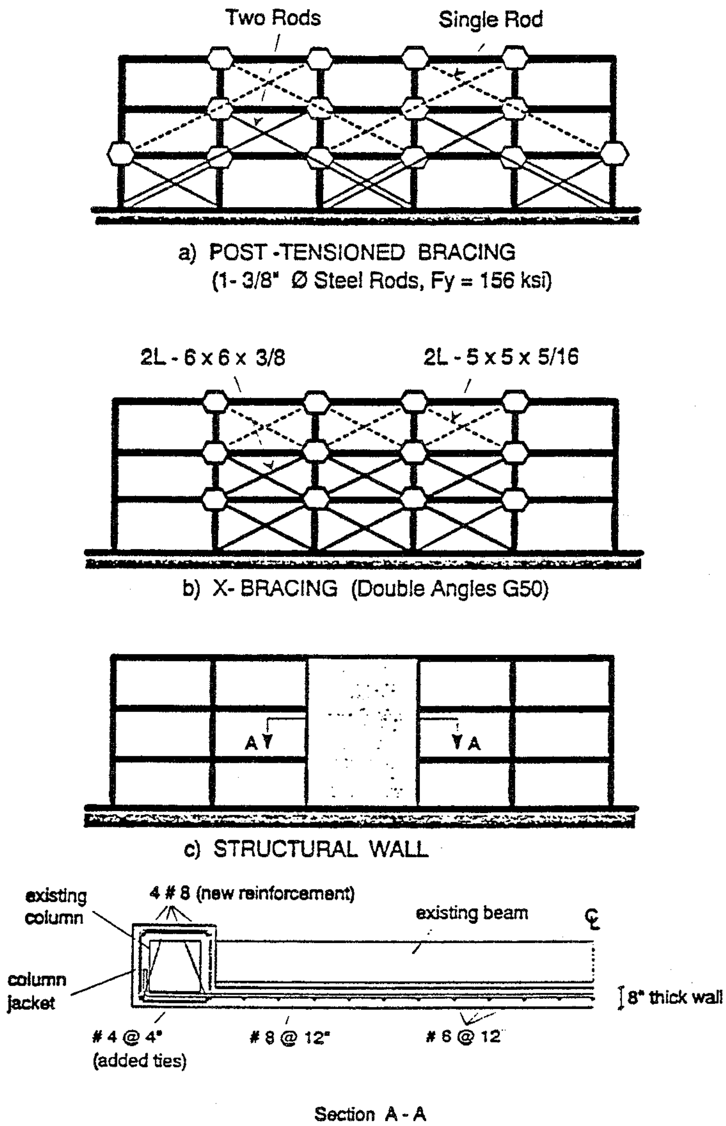
from Murray and Parker [1994]

Figure 11 Typical Shearwall Section



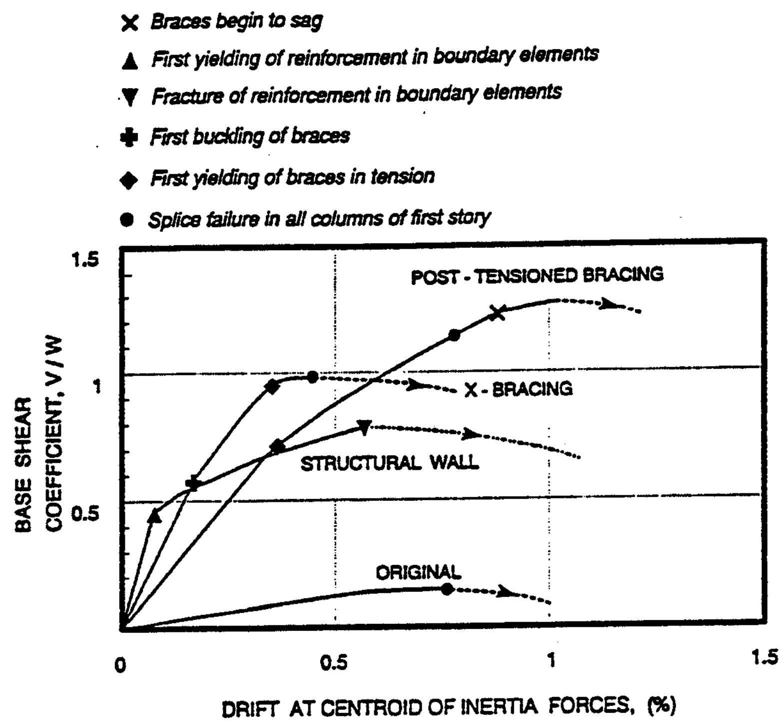
from Murray and Parker [1994]

Figure 12 Shearwall to Existing Column Detail



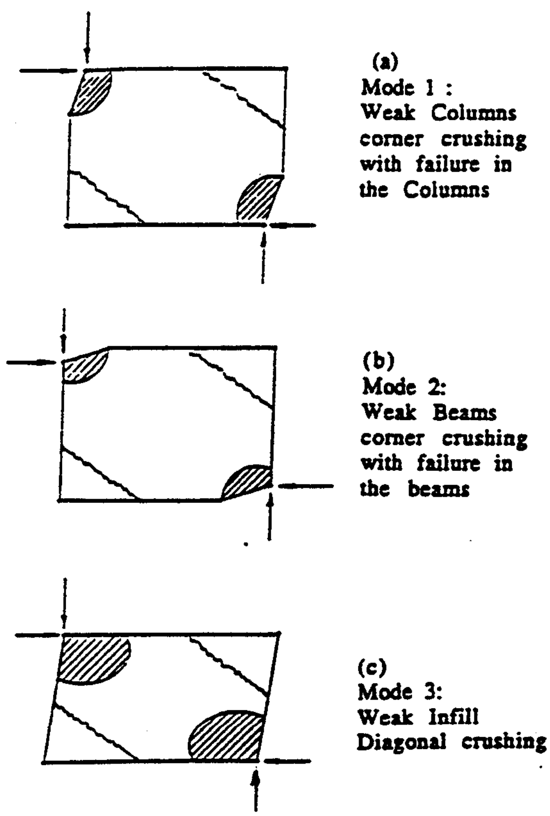
from Pincheira [1993]

Figure 13 Strengthening Schemes for the Three-Story Building



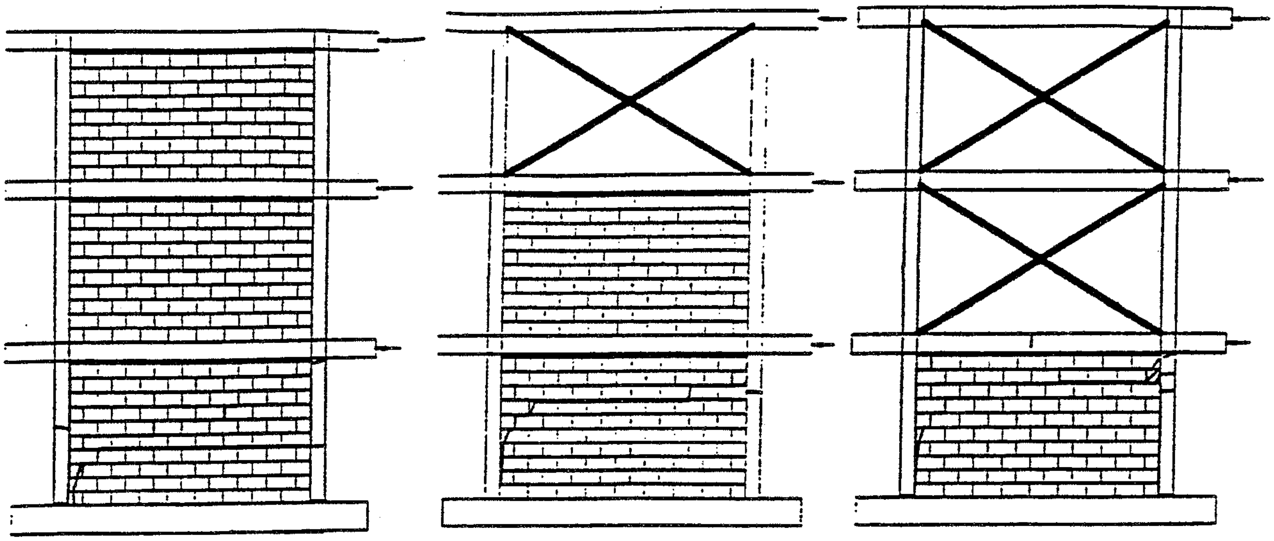
from Pincheira [1993]

Figure 14 Comparison of Base Shear Coefficient and Drift Relationship for the Original and Retrofitted Buildings



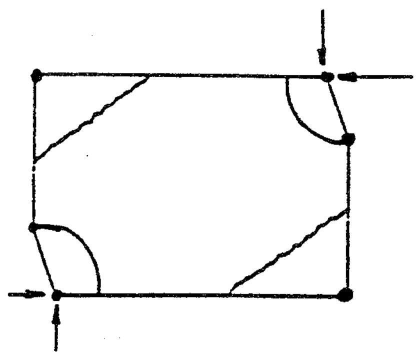
from Harris, Ballouz, and Kopatz [1993]

Figure 15 Collapse Modes for Single Story Non-Integral Infilled Frames

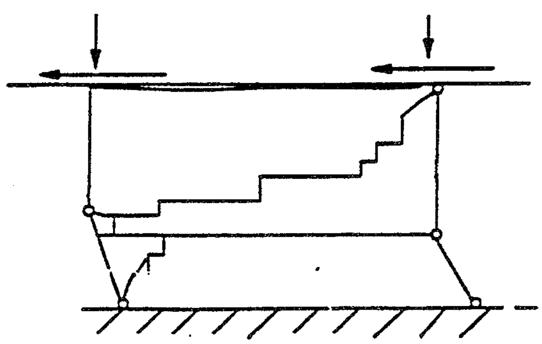


from Harris, Ballouz, and Kopatz [1993]

Figure 16 Failure Modes for Three Story Infilled LRC Frames

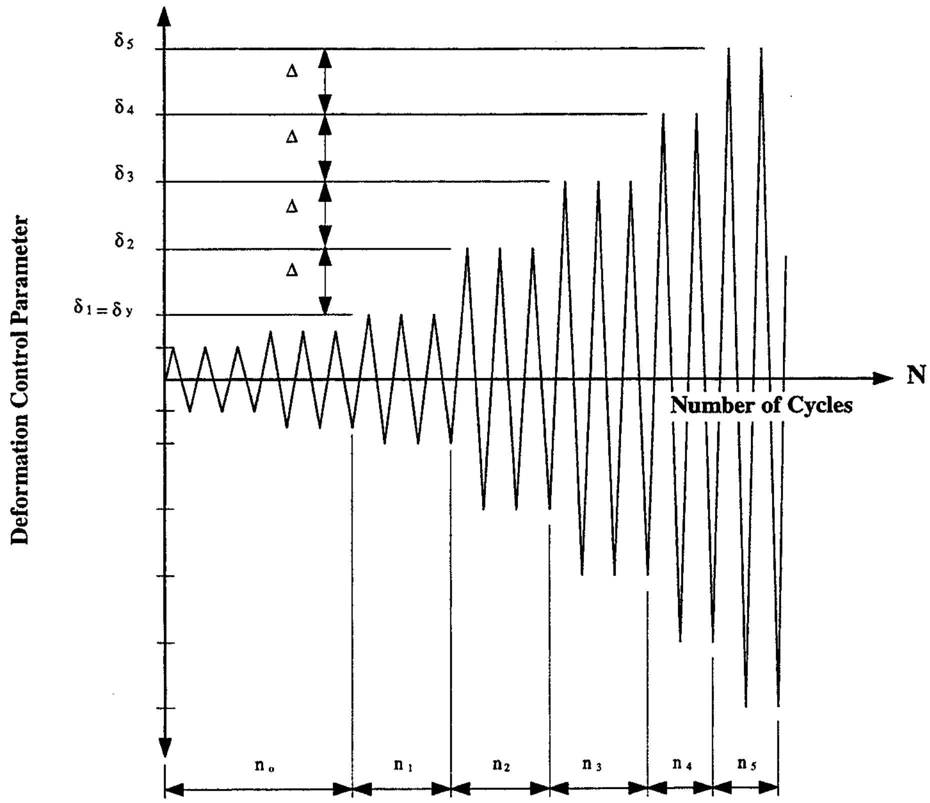


Mode 1 :
Weak columns/corner crushing
with failure in the columns.
(a) Theoretical Prediction



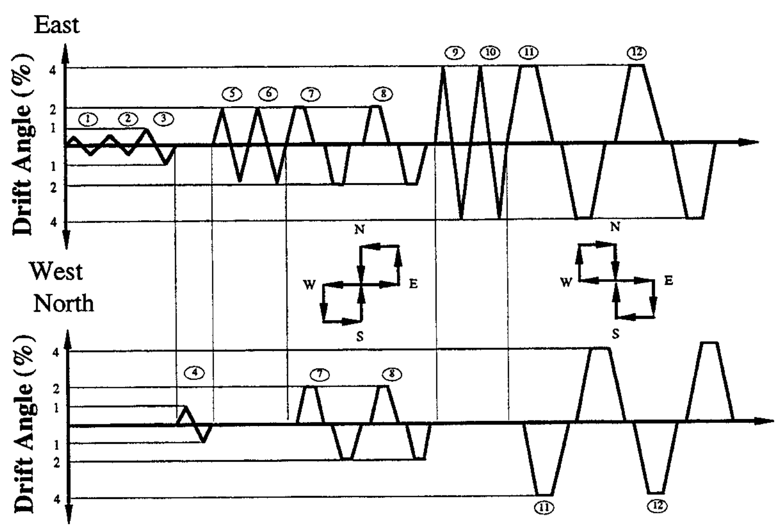
(b) Actual Failure Mode 3-Story Infilled LRCF.
from Harris, Ballouz, and Kopatz [1993]

Figure 17 Failure Mode = Weak Column Strong Beam (with Strong Infill)



from ATC [1992]

Figure 18 Deformation History for Multiple Step Test



from Alcocer and Jirsa [1990]

Figure 19 Bidirectional Cyclic Load History

INFLUENCE OF MASONRY INFILL ON LATERAL RESISTANCE OF REINFORCED CONCRETE FRAMES

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University of Colorado at Boulder

ABSTRACT

The behavior of a masonry-infilled R/C frame is very much influenced by the interaction between the infill and the bounding frame. To investigate this interaction, a number of single-story masonry-infilled R/C frames were tested, and the performance of a multi-story, multi-bay prototype frame with different frame and infill properties was assessed. Two types of R/C frames are considered in this study. One was designed for strong wind loads and the other for strong earthquake forces. Both were designed in accordance with current code provisions. The former, to a certain extent, also represents older structures that do not meet the detailing requirements of the current seismic design standards. The experimental results indicate that infill panels can improve the performance of R/C frames. Furthermore, specimens with strong frames and strong panels exhibited better performance than those with weak frames and weak panels in terms of the load resistance and energy-dissipation capability.

1.0 INTRODUCTION

Masonry infill panels are frequently used as interior and exterior partitions in reinforced concrete structures. They are usually treated as non-structural elements, and their interaction with the bounding frames is, therefore, often ignored in design. On the other hand, results of prior experimental studies (Fiorato et al. 1970; Klingner and Bertero 1976; Bertero and Brokken 1983; Zarnic and Tomazevic 1990) have indicated that infill panels can have a significant influence on the lateral resistance of a R/C frame. However, whether infill panels can lead to a better or worse structural performance has been a much debated issue. It depends, to a large extent, on the strength of the bounding frames with respect to those of the panels, the load resistance characteristics of the panels, and the configuration of the framing system. In spite of this controversy, masonry infill panels have been used as a means to strengthen existing moment-resisting frames in some countries, and there is evidence that they improved the performance of frame structures under severe earthquake loads (Amrhein et al. 1985).

The main difficulty in evaluating the performance of infilled structures is the interaction between the infill and the bounding frame, which could lead to a number of possible complicated failure mechanisms as illustrated in Fig. 1. A thorough understanding of this behavior requires extensive experimental investigations as well as applicable analytical tools. To address this issue, a comprehensive study was conducted at the University of Colorado, in conjunction with Atkinson-Noland & Associates, on the performance of masonry-infilled R/C frames subjected to in-plane lateral loads. This study focused on R/C frames that were designed in accordance with current code provisions, with and without the consideration of strong earthquake loadings. The objectives were to obtain experimental data on frame specimens that represented a large portion of the existing moment-resisting R/C frames, to develop detailed finite element analysis methods that can be used as research tools to understand the behavior of such structures, and to develop simple analysis methods that can be used in engineering practice. The details of this study can be found in the report by Mehrabi et al. (1994). It is hoped that the analysis methods developed in this study can be used as viable tools for the development of design recommendations on engineered infill, which can be used for the retrofit of existing R/C frame structures as well as in new construction.

The main focus of this paper is to present a concise summary of major experimental findings and their implications on the seismic performance of infilled frame structures. To this end, the experimental program and results are first summarized in the following two sections. The performance of a prototype structure is then evaluated based on different design scenarios that were considered in the experimental study. The limitations of this study are discussed and future research needs are identified in the concluding section of this paper.

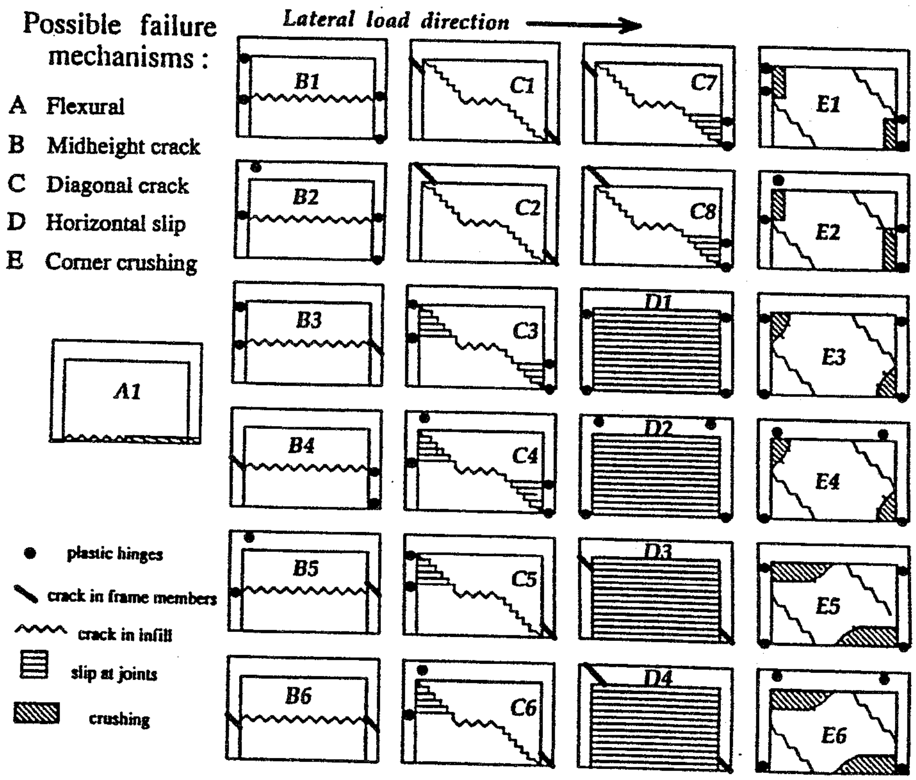


Figure 1. Failure Mechanisms of Infilled Frames

2.0 EXPERIMENTAL PROGRAM

The prototype frame selected in this study is shown in Fig. 2. It is a six-story, three-bay, moment-resisting R/C frame, with a 13.72-m-by-4.57-m (45-ft.-by-15-ft.) tributary floor area at each story. The design gravity loads complied to the specifications of the Uniform Building Code (UBC) (1991). The service live load was taken to be 2.39 kPa (50 psf), and the dead load was estimated to be 6.21 kPa (130 psf). For the purpose of parametric study, two types of frames were considered with respect to lateral loadings. One was a "weak" frame design, which was based on a lateral wind pressure of 1.24 kPa (26 psf), corresponding to a basic wind speed of 44.5 m/s (100 mph). The other was a "strong" frame design, which was based on the equivalent static forces stipulated for Seismic Zone 4 in the UBC. The former represented some existing R/C frames which do not have the detailing requirements that meet the current seismic design standards. The frames were designed in accordance with the provisions of ACI 318-89 (1989).

The test specimens were chosen to be 1/2-scale frame models representing the interior bay at the bottom story of the prototype frame. The design details for the weak and strong frames are shown in Fig. 3. The weak frames had weak columns and relatively strong beams, while the strong frames had relatively strong columns that had horizontal ties closely spaced near the ends. The beam design in a strong frame is identical to that in a weak frame, except that the former had more shear reinforcement in the critical regions. Each beam-to-column joint in a strong frame had four horizontal stirrups to prohibit brittle shear failure. While the strong frames had a height/length (h/l) ratio of about 1/1.5, two h/l ratios were considered for the weak frames. They were approximately 1/1.5 and 1/2.0. For infill panels, 0.1 x 0.1 x 0.2-m (4 x 4 x 8-in.) hollow and solid concrete masonry blocks were used in respective specimens. In each specimen, the infill panel was constructed by a professional mason after the frame had been cast. In the bed joints of the hollow blocks, mortar was applied onto the face shells only, whereas the solid blocks had mortar applied onto the entire bed joint. The head joints were partially filled with mortar. The bed and head joints were 9.5-mm (3/8-in.) thick.

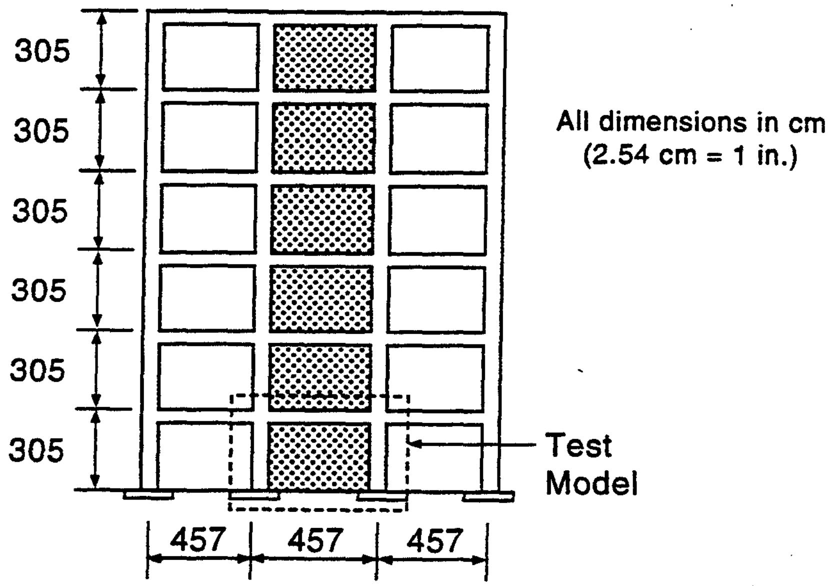


Figure 2. Prototype Frame

Twelve single-bay specimens were tested as shown in Table 1. Two additional tests were conducted on a two-bay frame, but they are not presented in this paper. Some of the frame specimens were tested more than once. In these cases, the cracks were repaired with epoxy injection, and the crushed regions were patched up with cement paste of strength com-

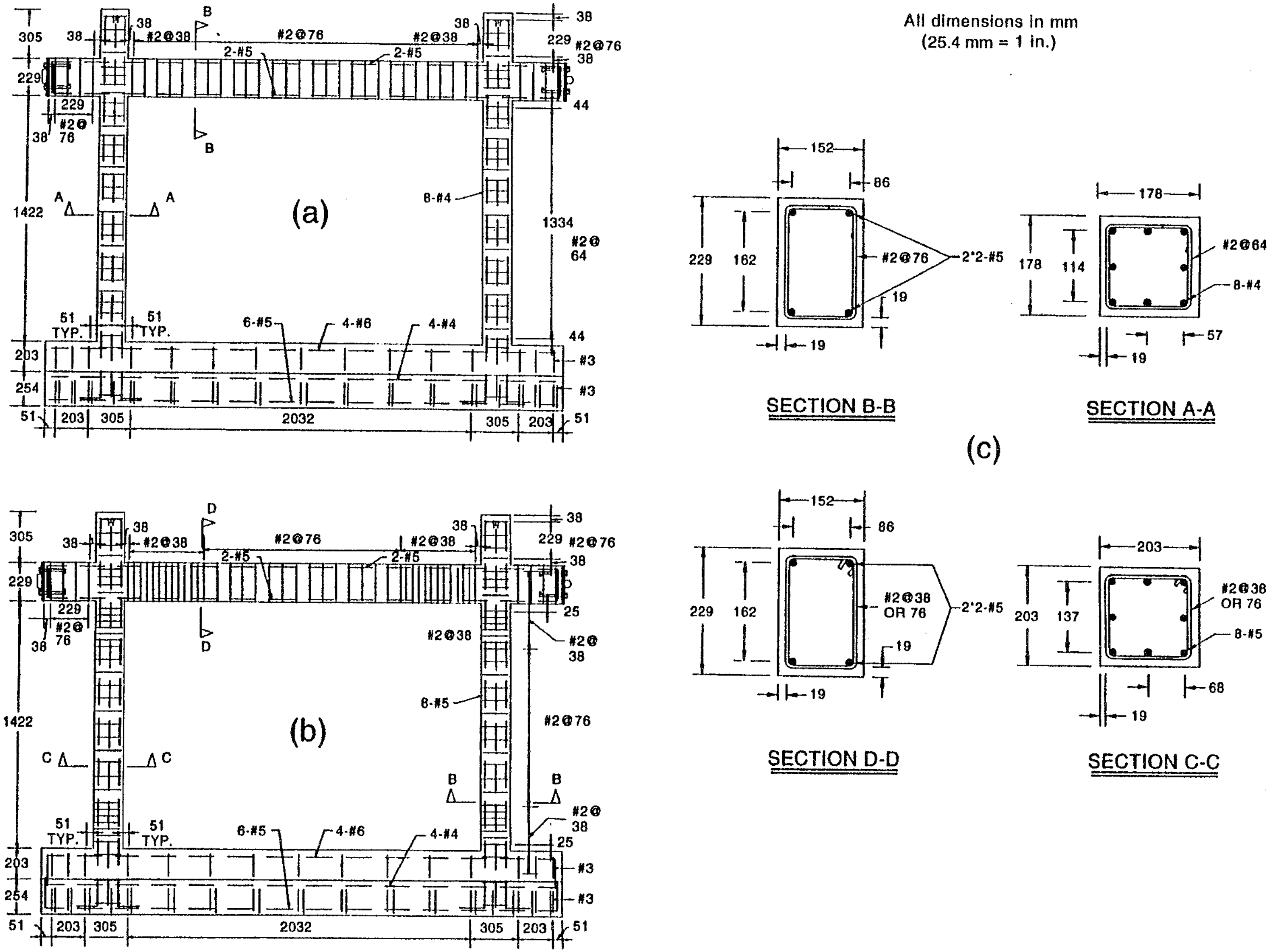


Figure 3. Design Details of Frame Specimens: (a) Weak Frame; (b) Strong Frame; (c) Member Cross Sections

parable to that of the original concrete. A new infill panel was used for each test. The specimens were tested at least 28 days after the construction of the infill. The lateral load was applied by means of two servo-controlled hydraulic actuators. The vertical loads were exerted by manually controlled hydraulic jacks, and they were maintained constant during each test. The specimens were subjected to different combinations of vertical and lateral loads. Two different vertical load distributions were employed: one with vertical loads applied onto the columns only, and the other with 1/3 of the total vertical load applied onto the beam and 2/3 onto the columns. Two types of in-plane lateral displacement histories were selected. They were monotonically increasing and cyclic.

Table 1. Test Specimens (1 kN = 0.225 kips)

Spec. No.	Type of Frame	Type of Masonry Units	Panel Aspect Ratio (h/l)	Lateral Load	Vertical Load Distribution (kN)	
					Columns	Beam
1	weak	no infill	0.67	monotonic	294	---
2	weak - repaired (1)*	hollow	0.67	monotonic	294	---
3	weak - repaired (2)*	solid	0.67	monotonic	294	---
4	weak	hollow	0.67	cyclic	196	98
5	weak	solid	0.67	cyclic	196	98
6	strong	hollow	0.67	cyclic	196	98
7	strong	solid	0.67	cyclic	196	98
8	weak - repaired (4)*	hollow	0.67	monotonic	196	98
9	weak - repaired (8)*	solid	0.67	monotonic	196	98
10	weak	hollow	0.48	cyclic	196	98
11	weak	solid	0.48	cyclic	196	98
12	weak - repaired (10)*	solid	0.48	cyclic	294	147

* Specimen repaired

3.0 EXPERIMENTAL RESULTS

3.1 Bare Frame

A weak bare frame was subjected to a monotonically increasing lateral load. It exhibited a fairly flexible and ductile behavior, as shown by the load-displacement curve in Fig. 4. However, severe shear cracks developed in the beam-to-column joints due to inadequate lateral reinforcement in these regions.

3.2 Infilled Frames

As shown in Fig. 4, infill panels increased the strength and stiffness of a R/C frame by a substantial amount. In an infilled frame, nonlinear behavior was usually started by the cracking of the infill. These cracks often initiated in the form of inclined cracks at the top compression corners with an approximately 45-degree angle. They were later joined by horizontal sliding cracks developed along the bed joints near the midheight of the panel. This type of cracks will be referred to as diagonal/sliding cracks in the following discussions.

The failure mechanism of an infilled frame depends very much on the relative strengths of the frame and the infill. As shown in Fig. 5(a), a frame with a weak (hollow) panel (Specimen 4) had its lateral resistance governed by the sliding of the bed joints often occurring over the entire panel. In such a case, the resistance of the panel does not seem to be influenced by the frame-panel interaction, and the total strength of the specimen is equal to the flexural resistance of a bare frame plus the sliding-shear strength of the panel (Mehrabi et al. 1994). In the case of a strong infill and a weak frame (Specimen 5), the ultimate resistance and failure were very much dominated by the diagonal/sliding crack and the shear failure of the windward column (see Fig. 5(b)). In the case of a strong infill and a strong frame (Specimen 7), the ultimate resistance was governed by the corner crushing in the infill (see Fig. 5(c)). In this case, the diagonal compression strut mechanism was fully developed, and the infill was very effective in enhancing the lateral resistance of the frame. This mechanism, which has been reported by Stafford Smith (1966) and others, depends very much on the frame-panel interaction. The lateral load-vs.-lateral displacement curves of these specimens are shown in Fig. 6. It can be observed that a stronger infill led to a higher lateral resistance as well as a better energy-dissipation capability. However, such an improvement was more pronounced in the strong frame (Specimen 7) than in the weak frame (Specimen 5) because of the brittle shear failure occurring in the columns of the latter.

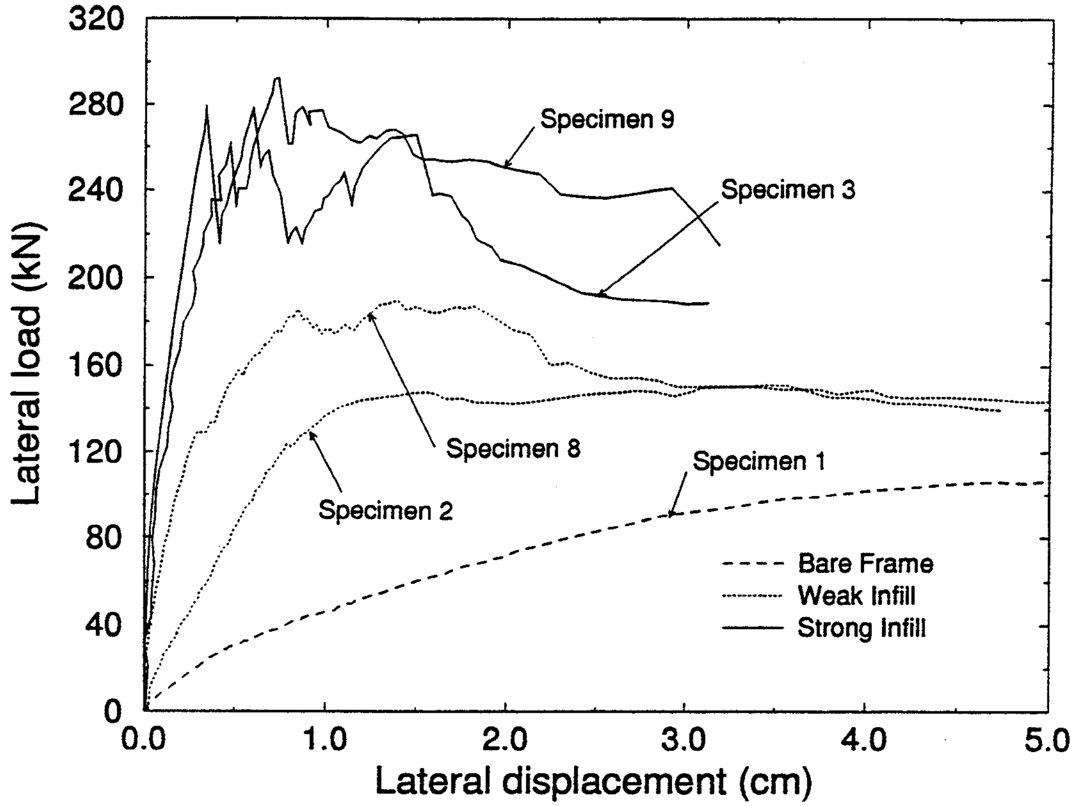
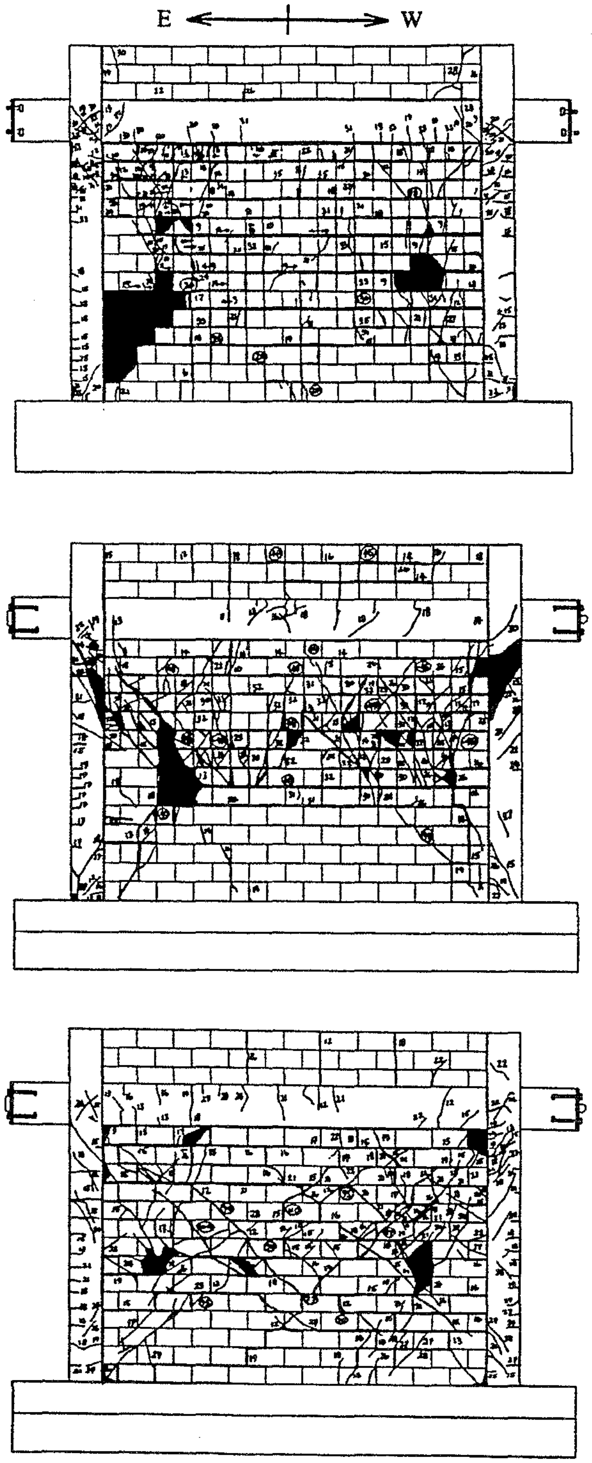


Figure 4. Specimens subject to Monotonically Increasing Loads
(1 cm = 0.394 in., 1 kN = 0.225 kips)



(a)

(b)

(c)

Figure 5. Failure Patterns of Infilled Frames: (a) Sliding of Bed Joints (Specimen 4); (b) Shear Failure of Columns (Specimen 5); (c) Crushing at Panel Corners (Specimen 7)

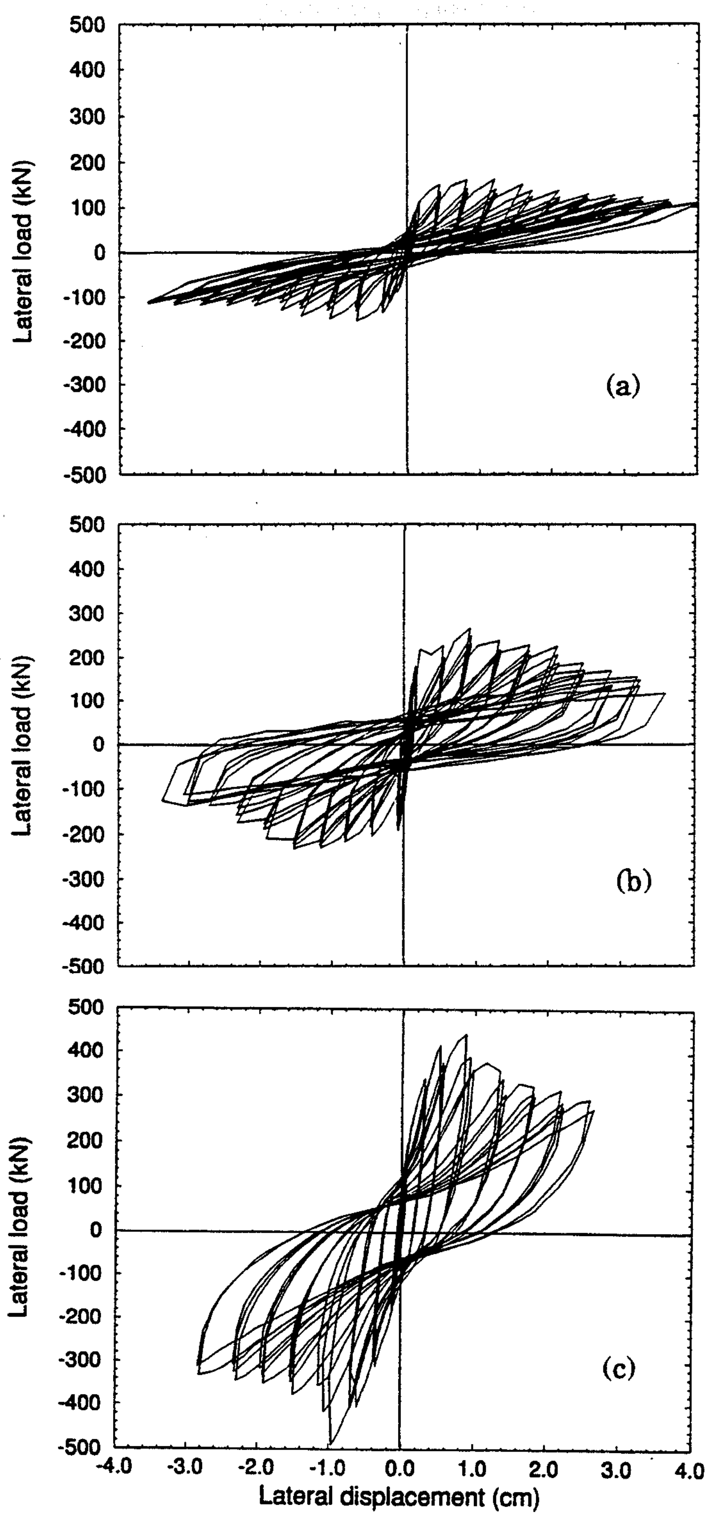


Figure 6. Specimens subject to Cyclic Loads: (a) Specimen 4 (Weak Frame-Weak Panel); (b) Specimen 5 (Weak Frame-Strong Panel); (c) Specimen 7 (Strong Frame-Strong Panel) (1 cm = 0.394 in., 1 kN = 0.225 kips)

The initial stiffness, critical loads, critical displacements, and failure mechanisms of the twelve specimens are summarized in Table 2. For specimens subjected to monotonically increasing loads, the secant stiffness is defined as the slope of the line joining the origin of the load-displacement curve and the point at which 50% of the maximum resistance is first reached. For the case of cyclic loadings, the secant stiffness is the slope of the line connecting the extreme points of a small-amplitude displacement cycle in which the peak load is about 50% of the maximum lateral resistance. As shown by Specimens 1, 4, and 5, the stiffness of a weak frame-weak panel specimen was about 15 times as large as that of the bare frame, while that of a weak frame-strong panel specimen was about 50 times as large. However, for the repaired specimens, such as Specimens 8 and 9, the increase in stiffness appeared to be much smaller. The maximum load resistance of a weak frame-weak panel specimen was about 1.5 times that of the bare frame, while the resistance of a weak frame-strong panel specimen was about 2.3 times. In spite of the fact that no test had been conducted on a strong bare frame, the lateral resistance of the strong frame was estimated to be 145 kN (32.7 kips). Comparing this to the strengths developed by Specimens 6 and 7 indicates that the maximum resistance of the strong frames was increased by the weak and strong infill panels by factors of 1.4 and 3.2, respectively.

Table 2. Critical Strengths of Test Specimens
(1 cm = 0.394 in., 1 kN = 0.225 kips)

Spec. No.	Secant Stiffness (kN/cm)	Load at First Major Crack in Panel *(kN)	Disp. at First Major Crack (cm)	Max. Lateral Load (kN)	Disp. at Max. Load (cm)	Failure Mechanism**
1	42	----	----	106.3	6.53	flexural
2	----	----	----	----	----	D1
3	1296	277.7	0.33	277.7	0.33	C7
4	753	-133.5, +92.1	-0.36, +0.10	-153.5, +162.4	-0.71, +1.19	D1 + E3
5	2242	-204.7, +218.9	-0.46, +0.20	-232.3, +267.0	-1.52, +0.91	C7
6	841	-182.9, +205.6	-0.48, +0.56	-188.2, +207.4	-0.89, +0.97	D1
7	2558	-401.4, +417.4	-0.61, +0.51	-489.5, +445.0	-1.14, +1.02	E3
8	578	133.5	0.36	190.0	1.40	C7 + E3
9	1034	261.2	0.51	292.8	0.74	C7
10	692	-156.2, +189.6	-0.61, +0.61	-156.2, +189.6	-0.61, +0.61	D1 + E3
11	2575	-262.5, +292.8	-0.58, +0.56	-275.9, +292.8	-1.70, +0.56	C7
12	3416	-332.0, +329.7	-0.48, +0.36	-355.5, +362.6	-0.81, +0.71	C7

* Diagonal/Sliding Crack

** Defined in Fig. 1

The resistance of an infilled frame was not sensitive to its aspect ratio within the range considered in this study. However, this conclusion may not be valid if the change of aspect ratio is so significant that the predominant failure mechanism is altered. Furthermore, the distribution of the vertical loads between the columns and the beam did not affect the resistance of an infilled frame very much. Nevertheless, increasing the total vertical load by 50% increased the stiffness by 30% and the maximum resistance by 25%, as shown by Specimens 11 and 12. In general, the specimens subjected to cyclic loads exhibited a lower resistance and a more rapid load degradation with respect to the level of lateral displacement than those subjected to monotonically increasing loads.

4.0 DUCTILITY AND ENERGY-DISSIPATION CAPABILITY

4.1 Ductility

For the infilled frames, nonlinear behavior was usually initiated by the separation at the frame-to-panel interface. This introduced a small reduction in the lateral stiffness. After that, the initiation of major cracks or compressive crushing in the infill induced the first significant nonlinear behavior. To assess the ductilities of the infilled frames, the load-deflection curve of a frame specimen is idealized by an elastic-perfectly plastic behavior, as shown in Fig. 7. For such an idealized system, ductility can be defined as the ratio of the maximum allowable displacement to the displacement at which the first yield occurs. In this study, two cases are considered in terms of the maximum allowable displacement. In one case, it is taken to be the displacement at which the maximum resistance is developed, Δ_{rm}, and, in the other, it is the displacement at which the lateral resistance is reduced to 80% of the maximum lateral resistance, Δ_{80}. The latter was the displacement at which the degradation of structural resistance began to accelerate in most of the infilled frame specimens. These ductility measures can be expressed as follows.

$$\mu_{rm} = \frac{\Delta_{rm}}{\Delta_y} \qquad \text{or} \qquad \mu_{80} = \frac{\Delta_{80}}{\Delta_y} \tag{1}$$

in which Δ_y is the yield displacement. The yield displacement is obtained by dividing the yield resistance, r_y, by the elastic stiffness, k. To be conservative, r_y is defined to be 80% of the maximum resistance, r_{max}, obtained from the tests, as shown in Fig. 7. The elastic stiffness, k, is approximated by a secant stiffness, which has been defined in the previous section.

For the bare frame, the assumed elastic-perfectly plastic behavior is very close to the actual response curve obtained from the test. Unlike infilled frames, the bare frame did not exhibit a significant load degradation after the maximum lateral resistance had been reached. For this reason, the maximum allowable drift for the bare frame is taken to be 2%, with respect to the story height, which is considered to be significant from the damage standpoint. It is interesting to point out that for the infilled frame specimens, the drift levels corresponding to the maximum allowable displacements defined previously are lower than 2% in all cases. Based on the 2% drift limit, the maximum allowable displacement for the bare frame specimen is 30 mm (1.2 in.).

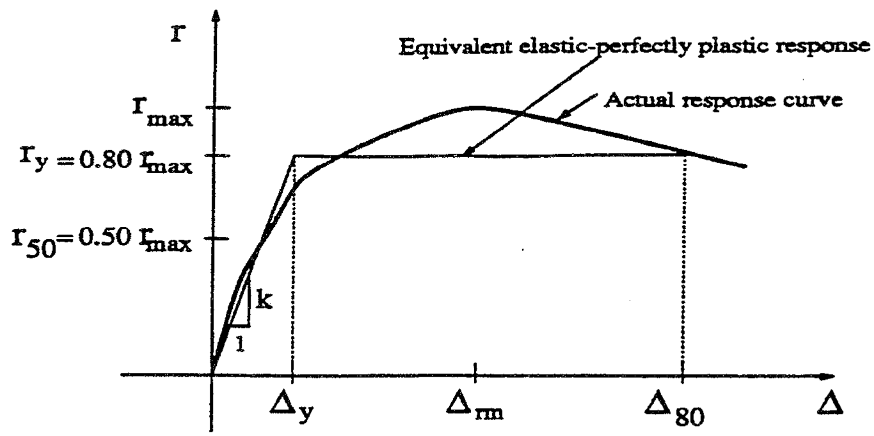


Figure 7. Idealized Elastic-Perfectly Plastic Load-Displacement Curve

The values of the idealized parameters and the ductilities computed for the frame specimens are shown in Table 3. The results indicate that the ductilities of the infilled frames are higher than that of the bare frame. This is due to the higher elastic stiffness and, consequently, the lower yield displacement of an infilled frame, when compared to those of the bare frame. In most cases, the specimens that had strong panels appear to be more ductile than those with weak panels, in spite of the fact that there were brittle shear failures developing in the weak columns. This is due to a considerable drop of the yield displacement caused by the strong panels. Furthermore, as shown by Specimen 12, increasing the vertical loads reduced the ductility. However, one should be aware of the fact that the results obtained from these single-story specimens may not be applicable to multi-story frames due to the absence of large overturning moments in these specimens. The extent of this influence depends on the slenderness ratio of the frame as well as the distribution of infill panels.

4.2 Energy-Dissipation Capability

The energy-dissipation capability of a frame specimen is defined as the total cumulative energy dissipated by the specimen when it reaches the ultimate limit state. As mentioned previously, two cases are considered for the ultimate limit state. It can be considered as the state at which the maximum lateral resistance is developed or the state at which the lateral resistance is reduced to 80% of the maximum resistance.

The cumulative energy dissipation calculated for the specimens is shown in Table 3. In general, those having a strong panel dissipated more energy than those having a weak panel. The specimens with the lower aspect ratio dissipated less energy at the maximum resistance and more energy at 80% of the maximum resistance than those with the higher aspect ratio. Increasing the vertical load resulted in a higher energy dissipation at the maximum resistance and a lower energy dissipation at 80% of the maximum resistance.

5.0 IMPLICATIONS ON SEISMIC PERFORMANCE

The seismic performance of a structure depends on a number of factors, such as the natural period of vibrations, the yield resistance, the ductility, and the energy-dissipation

Table 3. Ductility and Energy-Dissipation Capability of Frame Specimens
(1 cm = 0.394 in., 1 kN = 0.225 kips, 1 kJ = 8.85 in.-kip)

Test No.	Maximum Resistance r_{max} (kN)	Equivalent Elastic-Plastic System			At Maximum Resistance			At 80% of Maximum Resistance		
		Yield Resistance $r_y = 0.8 r_{max}$ (kN)	Stiffness k ** (kN/cm)	Yield Displacement Δ_y (cm)	Displacement Δ_{rm} (cm)	Ductility μ_{rm}	Energy Dissipated U (kJ)	Displacement Δ_{80} (cm)	Ductility μ_{80}	Energy Dissipated U (kJ)
1	106.4	85.0	42	2.032	---	---	---	3.05 +	1.5 +	1.36 +
2	---	---	---	---	---	---	---	---	---	---
3	277.7	222.0	1296	0.173	0.33	1.5	0.34	1.78	10.3	3.96
4	157.9 *	126.4	753	0.168	0.94	4.5	7.12	2.24	13.3	19.10
5	249.6 *	199.8	2242	0.089	1.22	10.9	9.94	2.18	24.6	32.77
6	198.0 *	158.4	841	0.188	0.91	3.9	5.20	2.74	14.6	28.59
7	467.3 *	373.8	2558	0.147	1.07	7.2	27.23	1.60	10.9	47.46
8	190.0	152.2	578	0.262	1.38	4.3	1.81	2.79	10.7	4.41
9	292.8	234.1	1034	0.226	0.74	2.6	1.02	3.05	13.5	7.12
10	173.1 *	138.4	692	0.201	0.61	2.4	2.49	2.89	14.4	32.65
11	289.3 *	231.4	2575	0.089	0.69	6.1	6.44	2.31	26.0	44.97
12	359.1 *	287.4	3416	0.084	0.76	7.3	10.51	1.57	18.8	22.37

* Average of two directions
** Based on the secant stiffness as defined in the text
\+ Based on 2% drift limit

capability, in addition to the characteristics of the earthquake ground motion. While some of these properties have been identified for the frame specimens, their actual performance is governed by a combination of these effects. To assess the performance of the prototype frame in a meaningful manner, elastic and inelastic response spectrum analyses are conducted with the frame properties deduced from the test results.

5.1 Equivalent Single-Degree-of-Freedom System

The prototype frame shown in Fig. 2 is idealized to have six lateral degrees of freedom. Its equations of motion can be expressed as

$$\underline{m}\,\ddot{\underline{v}} + \underline{r} = -\underline{m}\{1\}\ddot{v}_g \tag{2}$$

in which $\underline{m}$ is the mass matrix, $\underline{v}$ is the displacement vector, with each superposed dot representing differentiation with respect to time, $\underline{r}$ is the restoring force vector, $\{1\}$ is a unit vector, and $\ddot{v}_g$ is the ground acceleration. If one assumes that the response of the structure is dominated by its fundamental mode, i.e., $\underline{v} = \underline{\phi}_1 Y_1$, where $\underline{\phi}_1$ is the fundamental modal vector and Y_1 is the corresponding displacement coordinate, Eq. (2) can be reduced to

$$<1>\underline{m}\,\underline{\phi}_1 \ddot{Y}_1 + <1>\underline{r} = -<1>\underline{m}\{1\}\ddot{v}_g \tag{3}$$

in which $<1>$ is a unit row vector. By normalizing $\underline{\phi}_1$ in such a manner that the value for the first story is unity, i.e., $\phi_{11} = 1$, one has $Y_1 = v_1$, where v_1 is the displacement response at the first story. Consequently, Eq. (3) becomes

$$L_1 \ddot{v}_1 + V_o = -M_t \ddot{v}_g \tag{4}$$

in which $L_1 = <1>\underline{m}\,\underline{\phi}_1$, $M_t = <1>\underline{m}\{1\}$ is the total mass of the structure, and $V_o = <1>\underline{r}$ is the base shear, as shown in Fig. 8(a). Equation (4) represents the equation of motion for an equivalent single-degree-of-freedom system. For a linearly elastic system,

$$V_o = <1>\underline{k}\,\underline{\phi}_1 Y_1 = \omega_1^2 L_1 v_1 = K v_1 \tag{5}$$

in which $\underline{k}$ and ω_1 are the stiffness matrix and fundamental angular frequency of the structure, and $K = \omega^2 L_1$ is the elastic stiffness of the equivalent system. This approximation can also be applied to an inelastic structure by adopting a general V_o-vs.-v_1 relation for the structure. For this purpose, an elastic-perfectly plastic behavior is assumed for the equivalent system, as shown in Figure 8(b). The yield resistance V_{oy} can be determined from the experimental results presented previously as will be explained later on, while the yield displacement can be computed with the following relation.

$$v_{1y} = \frac{V_{oy}}{K} \tag{6}$$

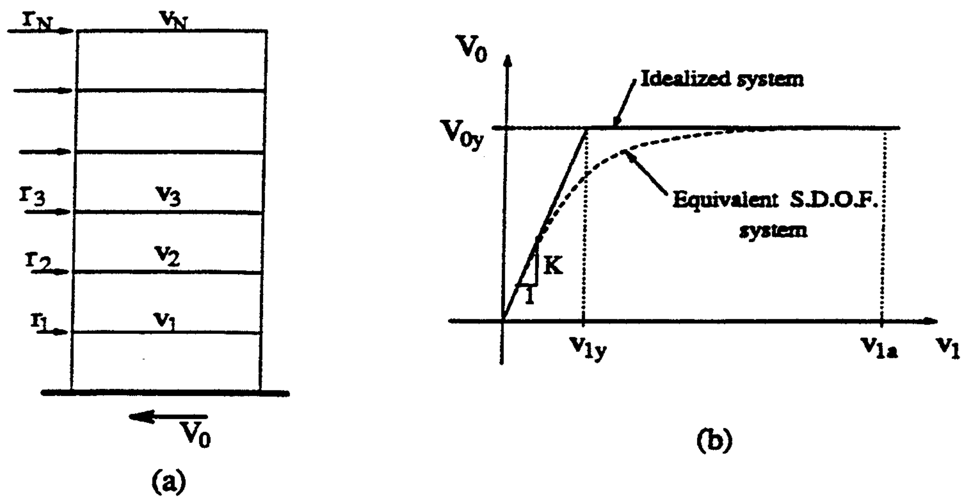


Figure 8. Equivalent Single-Degree-of-Freedom System: (a) Structure; (b) Base Shear vs. First-Story Displacement

According to Eq. (1), the ductility factor for the equivalent system is defined as

$$\mu = \frac{v_{1a}}{v_{1y}} \tag{7}$$

in which the maximum allowable displacement for the equivalent system, v_{1a}, can be estimated from the experimental results from the single-story frames.

For response spectrum analyses, the elastic and inelastic properties of the prototype frame and of the corresponding equivalent system are extracted from the experimental results in the following manner.

5.2 Elastic Properties of Prototype Frame

Analyses have been performed with SAP90 (Habibullah and Wilson 1991) to obtain the modal vector, $\underline{\phi}_1$, and the fundamental period, T, of the prototype frame with and without infill panels. However, the comparison of the experimental and analysis results has shown that the actual stiffness of a R/C frame is much lower than the theoretical stiffness com-

Table 4. Maximum Allowable Ground Accelerations for the Prototype Frame (1 cm = 0.394 in., 1 kN = 0.225 kips)

Specimens					Equivalent SDOF System for Prototype Frame													
									At Max. Resistance		At 80% of Resistance		Max. Allowable Peak Ground Acceleration (g)					
													$V_{1a} = V_{1y}$		$V_{1a} = V_{rm}$		$V_{1a} = V_{80}$	
Test No.	Frame	Aspect Ratio	Vert. Load (kN)	Infill	Period T (sec)	K (kN/cm)	V_{0y} (kN)	V_{1y} (cm)	V_{rm} (cm)	μ_{rm}	V_{80} (cm)	μ_{80}	Sylmar	El Centro	Sylmar	El Centro	Sylmar	El Centro
1	weak	0.67	294	no	1.50	149	685	4.60	--	--	6.10	1.3	0.45	0.55	--	--	0.60 *	0.70 *
4			294	weak	0.54	1160	547	0.47	1.88	4.0	4.47	9.5	0.10	0.10	0.35	0.45	0.55	0.90
5			294	strong	0.41	2099	841	0.40	2.44	6.1	4.37	10.9	0.20	0.20	0.45	0.65	0.80	1.00
--	strong	0.67	294	no	1.32	188	846	4.49	--	--	6.10	1.4	0.55	0.55	--	--	0.50 *	0.70 *
6			294	weak	0.49	1408	690	0.49	1.83	3.7	5.49	11.2	0.15	0.15	0.40	0.45	0.75	1.20
7			294	strong	0.37	2577	1553	0.60	2.08	3.5	3.20	5.3	0.35	0.35	0.75	0.75	1.00	1.00
10	weak	0.48	294	weak	0.50	1452	596	0.41	1.22	3.0	5.79	14.2	0.10	0.10	0.25	0.30	0.70	1.15
11			294	strong	0.33	3562	970	0.27	1.37	5.0	4.62	17.0	0.20	0.20	0.55	0.50	0.85	1.20
12			440	strong	0.33	3562	1193	0.33	1.52	4.5	3.15	9.5	0.25	0.25	0.65	0.60	0.85	0.95

* Based on 2% drift limit

puted with uncracked concrete sections. Hence, an effective moment of inertia, I_e, which is assumed to be 44% of that of an uncracked section, has been adopted in the eigenvalue analyses. This is determined from the experimental results. For the prototype frame with infill, it is assumed that only the center bay has infill panels, and the entire infilled bay is modeled with an equivalent beam, whose stiffness is estimated from that of an infilled frame specimen. The natural period of the prototype frame which represents each specimen design is computed and shown in Table 4.

The elastic stiffness, K, of an equivalent single-degree-of-freedom system, as shown in Table 4, is computed with the relation in Eq. (5), in which L_1 is based on the eigenvectors obtained from the aforementioned analyses.

5.3 Inelastic Properties of Prototype Frame

The yield base shear, V_{oy}, which represents the maximum resistance, is influenced, to a large extent, by the failure mechanism of the prototype frame. To evaluate this, the failure mechanisms shown in Fig. 9 are assumed. For a weak bare frame and a frame with infill panels, it is assumed that a soft-story mechanism (Figs. 9(b) and 9(c)) will dominate. For a strong bare frame, which has a strong column-weak beam design, it is assumed that the collapse load is governed by the plastic moments developed in the beams (Fig. 9(a)). With these assumptions, the inelastic response of a strong bare frame can be approximated by a linear modal vector, and those of a weak bare frame and infilled frame by a uniform modal vector, as shown in Fig. 10.

To obtain the yield base shear for the strong bare frame, a collapse analysis is conducted with the failure mechanism shown in Fig. 9(a). For this analysis, the lateral forces are assumed to be linearly proportional to the height of the structure, and the plastic moment capacities of the frame members are calculated with the procedure recommended in the ACI code (1989). However, the calculated moment capacities are increased by 15% to account for the possible discrepancy between the actual and theoretical values, as reflected by Specimen 1. To be conservative, the yield base shear, V_{oy}, is taken to be 80% of the maximum base shear computed.

In the case of the weak bare frame and infilled frames, where plastic deformation is concentrated in the first story, the yield base shear, V_{oy}, is considered to be equal to the yield resistance of the first story alone. For the weak bare frame, the yield resistance of the first story is taken to be twice as large as that of the middle bay, with the assumption that the interior and exterior columns are identical. The yield resistance of the middle bay of the prototype frame is obtained from that of Specimen 1 by applying a scale factor of 4 in accordance with the similitude rules. To obtain the yield resistance of the first story of an infilled frame, the lateral forces resisted by the exterior columns are added to the yield resistance of the middle bay. The yield resistance of the middle bay of the prototype frame is estimated from that of a test specimen. These values are shown in Table 4.

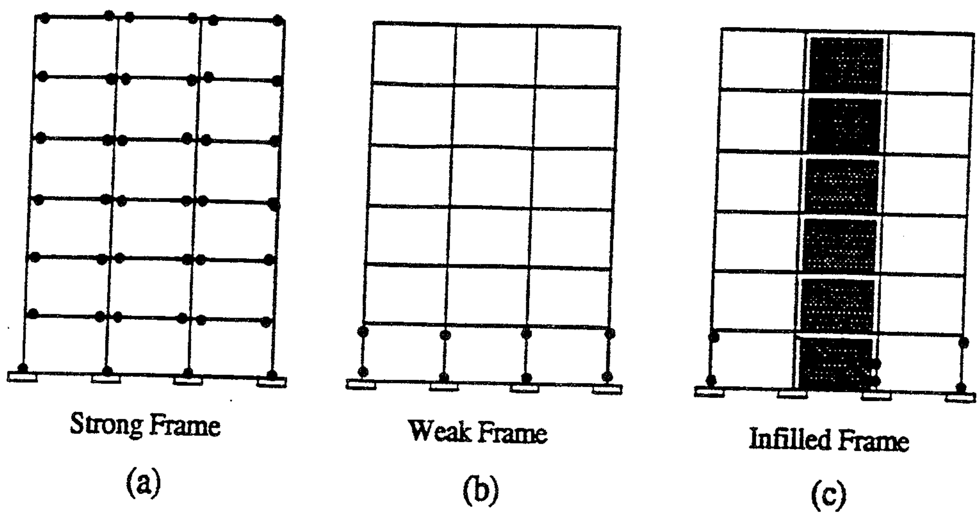


Figure 9. Failure Mechanisms of Prototype Frame

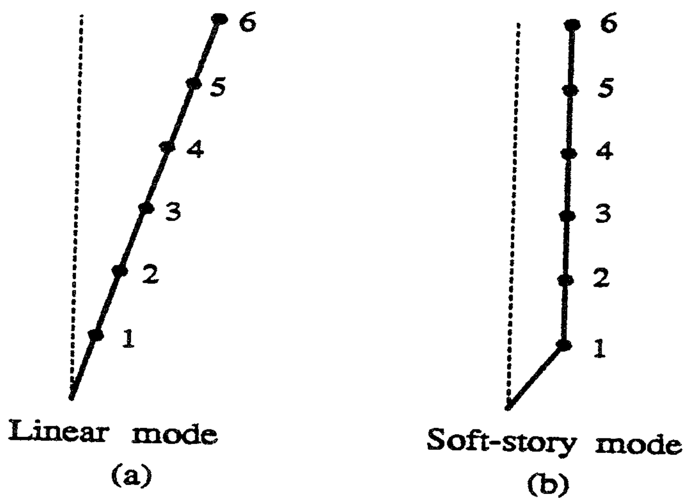


Figure 10. Idealized Mode Shapes for Inelastic Response

To compute the ductility of an equivalent single-degree-of-freedom system that represents a bare frame, v_{1a} is calculated with the 2% drift limit as discussed before. For an infilled frame, v_{1a} is assumed to be controlled by the middle bay. Hence, it can be obtained from the experimental results (see Table 3) by applying a scale factor of 2 in accordance with the similitude rules. However, as pointed out previously, because of the possibility of having a large overturing moment at the bottom story of a multi-story frame, such an extrapolation may not be conservative in certain cases. The maximum allowable displacements and the corresponding ductilities, μ_{rm} and μ_{80}, which are based on the maximum resistance and 80% of the maximum, are shown in Table 4. The yield displacements are computed with Eq. (6).

5.4 Response Spectrum Analysis

The NS component of the 1940 El Centro record and the EW component of the Sylmar record obtained in the 1994 Northridge earthquake are used to assess the performance of the prototype frame, which is idealized as an equivalent single-degree-of-freedom system. A damping ratio of 5% of the critical is adopted for all cases. Both elastic and inelastic response spectrum analyses are conducted. For the inelastic analyses, ductility spectra of the form presented by Bertero et al. (1978) are derived and used. The hysteresis curves used for these analyses are shown in Fig. 11. They are intended to mimic the experimental behavior shown in Fig. 6. The total mass M_t used in the analyses varies slightly from case to case depending on the size of the frame members and the type of masonry panels. The average value of M_t is 1.78 ton (1.46 kip sec^2/in.). The structures are evaluated by assessing the maximum ground accelerations which they can withstand without exceeding a prescribed limit state. The maximum allowable ground accelerations obtained for the different limit states are shown in Table 4.

The comparison of the results in Table 4 shows that the addition of infill panels tends to reduce the maximum ground acceleration that can be resisted by the structure before reaching the "elastic" limit state defined here. This is partly due to the fact that an infilled structure attracts more seismic forces, and partly due to the large yield displacement computed for the bare frame. It must be mentioned that, in reality, damage had already occurred in the bare frame specimen before the calculated yield displacement was reached. Furthermore, the maximum ground acceleration that can be resisted by an infilled frame before reaching the maximum resistance is, in most cases, less than that permitted by a bare frame based on the "elastic" limit state. For the ultimate limit state corresponding to 80% of the maximum resistance, the addition of infill panels is clearly beneficial in all cases. In general, a stronger panel results in a better performance.

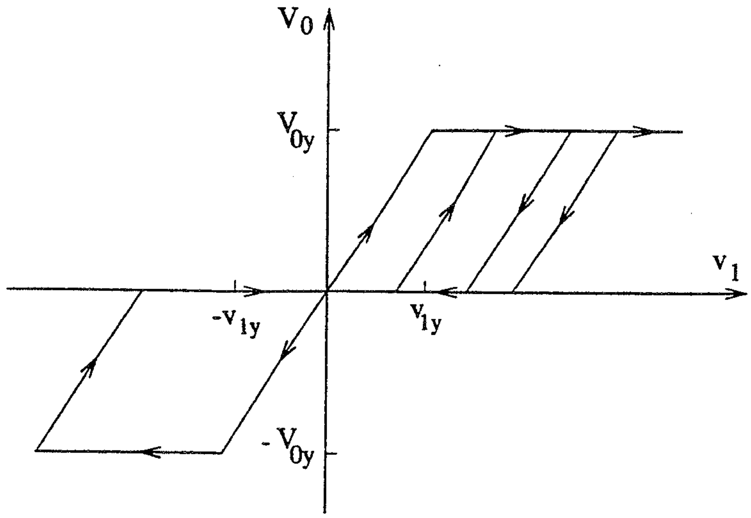


Figure 11. Idealized Hysteresis Curves used for Inelastic Analyses

It must be pointed out that the results obtained here are based on the assumption that the ductility of the multi-story frame can be directly deduced from that of a single-story frame. Nevertheless, ductility could be severely reduced in a slender multi-story frame (Fiorato et al. 1970). Even though this is not the case here, it warrants further investigation.

6.0 CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

6.1 Conclusions

Two types of frame specimens are considered in this study. One was designed for strong wind loads and the other for severe earthquake forces. The former, to a certain extent, also represents older structures that do not meet the detailing requirements of the current seismic design standards. Both solid and hollow concrete masonry units, which represented strong and weak infill panels, were used for the infill. The experimental results indicate that infill panels can improve the performance of R/C frames. Furthermore, specimens with strong frames and strong panels exhibited a better performance than those with weak frames and weak panels in terms of the load resistance and energy-dissipation capability.

In specimens with weak frames and strong panels, brittle shear failure was observed in the columns. Nevertheless, this generally occurred at relatively large drift levels, which were beyond 1% in most cases. These specimens also exhibited a relatively good energy-dissipation capability when compared to a weak frame-weak panel specimen. However, the main drawback of this type of failure is that it will jeopardize the stability of the structure, and that such damage is not repairable. In any case, the lateral loads developed by the infilled frames were consistently higher than that of the bare frame. This is even true for the least ductile specimen deforming up to a drift level of 2%.

However, one should be cautious about extending the above observations to other frame configurations. For example, experimental results by others have indicated that the ductility of a slender multi-story frame could be severely jeopardized by infill panels due to the large overturning moments that may occur at the bottom story. In such a case, the brittle shear failure of the columns could lead to an immediate loss of lateral load resistance.

6.2 Future Research

This study indicates that masonry infill panels can be potentially used as an effective means to improve the performance of R/C frames that do not satisfy the current seismic design standards and also be used as main load resisting elements in new R/C structures. However, rigorous engineering guidelines are yet to be developed for these purposes. To achieve this goal, the following items are recommended for future research.

1. Additional parametric studies are required to develop design rules that can lead to a desirable frame-panel interaction mechanism. In particular, rules for determining the optimal strength of infill panels with respect to those of the bounding columns are necessary. These parametric studies can be conducted numerically by means of finite element models.

2. It has been shown that a weak infill may not provide a good energy-dissipation capability. Hence, methods for enhancing such a capability without leading to a substantial increase in panel strength should be investigated. These include the possibilities of using light reinforcing steel and fiber-reinforced grout in hollow masonry panels.

3. Efficient strengthening methods should be developed to enhance the shear resistance of nonductile columns to avoid irreparable damage and catastrophic failure.

4. The influence of window and door openings in infill panels should be further investigated. Experimental data on this are very limited.

5. Studies should be conducted to examine the influence of frame configurations on the performance of infill frames. All design recommendations should be verified with multi-story, multi-bay frames of different slenderness ratios. This can be most efficiently investigated by means of finite element models with some experimental verifications.

6. Simple analytical tools that encompass the wide variety of possible failure mechanisms of infilled frames should be developed to assist in the design and performance evaluation of these structures.

ACKNOWLEDGEMENTS

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SEISMIC UPGRADING OF REINFORCED CONCRETE FRAMES WITH STEEL ELEMENTS

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ABSTRACT

This paper briefly reviews conceptual strategies and commonly used schemes for upgrading seismically deficient reinforced concrete frames by using steel elements. Also presented in the paper is the most related research work carried out in Japan and the U.S., as well as some suggestions for further research and implementation for improved design practice.

1.0 INTRODUCTION

There exists a large inventory of older reinforced concrete building structures in regions of varying seismicity in the U.S. and throughout the world which were not designed or detailed to resist seismic forces. In view of the advancing state of practice these structures can be considered quite deficient and represent serious hazard to life safety and potential for huge economic losses in future major earthquakes. The need for strengthening or upgrading those structures for survival during future earthquakes cannot be over-emphasized, especially, in view of the damage and life loss incurred during some recent earthquakes, such as the Mexico City earthquake of September 1985, the Loma Prieta earthquake of 1989, the Northridge earthquake of 1994 and the most recent Kobe earthquake of January 1995.

Efforts to repair and strengthening older reinforced concrete buildings in Mexico City go back to 1979. These efforts gathered renewed momentum after the 1985 disaster. The 1985 experience of Mexico City prompted similar efforts in the U.S. which were somewhat speeded up after the 1989 Loma Prieta earthquake. Japan appears to have more experience in this area where the efforts go back to the 1968 Tokachi-Oki earthquake. A variety of techniques for repair and strengthening have been employed by engineers. The more commonly used methods include: addition of concrete or masonry walls, and strengthening by external structural steel elements.

Reinforced concrete and steel have been the two most commonly used materials for strengthening thus far. Reinforced concrete has been used in the form of jacketing of existing frame columns or to add infilled shear walls. An extensive strengthening work with concrete often requires extended construction work and complete evacuation for longer periods of time. The weight of added concrete results in considerable increased inertia mass as well as increased foundation upgrading work. Infill concrete walls may also interfere with building function or limit openings. The main advantages of strengthening by steel systems are small increases in weight and much shorter construction time so that the strengthening work can be accomplished without major disruption of building occupancy.

This paper presents a brief review of some of the most commonly used systems for strengthening by added steel elements, some related research work carried out in Japan and U.S., as well as a few suggestions for needed research in this area.

2.0 CONCEPTS FOR SEISMIC STRENGTHENING

Older structures generally lack the following attributes which are considered to be essential for earthquake resistance:

(1) Lateral strength and stiffness
(2) Ductility
(3) Energy dissipation mechanism
(4) A combination of the above.

Several possible conceptual strategies can be used to improve the seismic performance of deficient structures. These are illustrated in Fig. 1.

One strategy can be to primarily increase the stiffness of the structure with the main goal of shortening the natural period. This strategy works best if the modified period takes the structure into the lower response spectrum region, such as, when ground conditions have associated dominant periods. Shortening of the natural period of the structure generally puts it into significantly increased strength demands which then has to be provided. This is commonly associated with any scheme for increasing stiffness. The overall goal in this philosophy is to reduce the displacement response of the modified structure. This is generally achieved by adding concrete shear walls or steel braced walls or shear panels.

It is also possible to achieve reduced displacement response by adding some type of energy dissipating mechanisms or devices without substantial increase in stiffness or strength. This scheme has the advantage that the structural response can be improved without requiring strengthening of the structure or the foundation. Moreover, in the event of severe ground motion, damage to the main frame is minimal and the devices could be replaced, if needed, with little effort. Alternatively, the same objective could be met by increasing the ductility and energy dissipation capacity of the primary lateral force resisting elements of the existing frame, such as the columns.

It is, however, more common to adopt a scheme which utilizes a balanced combination of increasing the strength, stiffness, ductility and energy dissipation capacity of the structure.

A review of some schemes that have been used by engineers in past or current practice for upgrading RC frames by using structural steel is presented in this section.

2.1 Steel Jackets and Plates

One of the most common deficiencies in older RC building frames is lack of strength and ductility of columns due mainly to inadequate shear reinforcement or inadequate splices of longitudinal reinforcing bars at critical locations, especially where plastic hinges may form, such as at column base or above floor beams and slabs. External steel jacketing is a very effective method to remedy such deficiencies by providing confinement to transverse or longitudinal reinforcement, as needed. This has been a very popular method of introducing ductility in column piers in RC bridges. In most cases increase in strength and ductility due to confinement alone may be adequate so that composite action may not be necessary. In those cases a small gap is left at the ends of the jacketing steel. Where the jacketing steel is also needed for additional composite strength it is necessary to provide continuity at the ends. The jacketing may not be needed over full length of the columns if the shear strength of the unjacketed column alone is sufficient in those portions. Some common jacketing types are shown in Fig. 2. They include solid plates in circular, oval or rectangular shapes with non-shrink mortar fill between the RC column and the steel. Circular or oval shapes provide better confinement. Sometimes the steel plates of rectangular jacket shape need to be anchored to the column by means of grouted studs in order to enhance their effectiveness to provide confinement or composite action. Alternatively, steel jackets can also be made of vertical angles, plates or channel shapes which are tied together by transverse bands or lattice bars welded to the vertical elements. When found adequate, transverse steel bands alone or longitudinal plates attached to the column by epoxy-grouted bolts or by epoxy bond are also used.

Steel jacketing can also be used for beams and slabs to provide increased strength, especially, in negative moment regions where adequate reinforcement may not be present because the frame was designed for gravity loads only. In those situations the beam and slab jacketing needs to be connected to the jacketing steel of columns. This method also helps the beam-to-column joints in transferring moments. Many times beam-to-column joints also have inadequate shear strength due to insufficient transverse reinforcement. Steel jackets can also enhance shear strength and ductility through added strength of steel as well as through confinement of existing concrete.

2.2 Bracing Systems

Concentric steel bracing is perhaps the most efficient structural system for resisting lateral forces because it provides complete truss action. Therefore, addition of steel bracing systems has been a very popular method for upgrading seismically inadequate RC frames. Steel bracing systems can be designed to provide stiffness, strength, ductility, energy dissipation, or any combination of these. Objectives ranging from pure drift control to collapse prevention can be achieved. The bracing system can be added to the existing structure either on the exterior frames or interior frames without disrupting the building occupancy too much. The strengthening system can be either attached to the existing frames piece by piece (as shown in Fig. 3) or can be prefabricated as infill panels and then attached to the frame (Fig. 4). The latter method has been quite popular in Japanese practice. In the first scheme, the steel elements of the bracing system are attached to the frame members through epoxy-grouted bolts or dowels and the bracing members themselves are then connected to the horizontal and vertical steel elements (Fig. 5). The second method employs prefabricated braced frame panels with welded shear studs on the outer periphery. A second set of shear studs are inserted in the RC frame at proper spacing and position. The braced panel is then set in position and the space surrounding the shear studs is packed with non-shrink mortar. A steel spiral is often placed between the two sets of studs in order to confine the mortar (Fig. 6).

Use of eccentric bracing and pre-tensioned cables has also been made, albeit, not too commonly.

2.3 Infill Steel Panels

Use of infill steel panels has also been made as an alternative to bracing system as was shown in Fig. 4. The methods of attaching the steel panel walls to the existing frame are similar to those employed for bracing systems. The steel panels may have openings for windows or doors. However, they commonly require stiffening against local buckling by stiffeners or shotcreting.

2.4 Damping

Use of added damping devices is a relatively recent development for upgrading existing structures. This method has the advantage that response can be reduced without any significant change in stiffness or period of the structure. The damping devices can be based on friction, or viscoelastic action, or inelastic hysteretic action. The latter ones can be made of short steel stub sections with inelastic action occurring in a web panel, or through inelastic flexure of an assembly of steel plates. These devices usually are inserted between the existing floor beam or slab and added steel bracing in a chevron pattern which are designed not to buckle or yield. Such systems can also be attached either piece by piece or as preassembled panels. This method has distinct advantage that the devices can be considered as "disposable" and easily replaced after a major event. An example is shown in Fig. 7.

3.0 RELATED RESEARCH

Many RC building structures have been upgraded during the last ten years or so in the U.S., Japan, and Mexico by using the concepts as outlined in the preceding section. While substantial research work has been carried out in the U.S. and Japan, there are no standard procedures for design and detailing of the strengthening systems. A vast majority of current practice is based mainly on engineering judgement using many conservative assumptions. A brief review of related research work on the subject is given in this section.

3.1 Steel Jackets and Plates

Steel jacketing has been a very popular method for increasing the ductility and/or shear capacity of RC columns in older building and bridge structures. Studies by Estrada [1] and Aboutaha et al.[2,3] have shown that full or partial encasement by steel jacketing can be very effective with careful attention to detailing the connection between the column and steel jacket. Among studies related to bridge column piers the latest work by Stojadinovic [4] on bridge outrigger columns has also produced excellent results. Study by Valluvan [5] showed that use of external angles and transverse straps can also produce good confinement effect on column lap splices. Study by Estrada [1] on the effect of attaching steel plates to remedy the deficiency of inadequate continuity of beam reinforcing bars into the column joint produced excellent results. Similar success was achieved by Stojadinovic [4] in his tests of bridge outrigger beams provided with steel jacketing.

3.2 Infill Panels and Bracing Systems

Strengthening of RC frames by using steel panels and bracing systems as infills has been a very popular method in Japanese practice. References by Yamamoto and Aoyama [6], Sugano and Fujimura [7], and Shimizu and Yamamoto [8] give excellent summaries of Japanese experimental and analytical research on this topic. Most of the Japanese work involved testing of reduced scale single bay, single story specimens. The results have shown effectiveness of this method with proper detailing of connections between the steel brace or plate panels and the RC frame members.

Mexican and U.S. practice has favored the use of steel bracing systems as piece-by-piece attachment to existing RC frames, which involves significant welding to be done on site. Tests and associated analytical work carried out by Jones and Jirsa [9], Bush et al. [10], and Badoux and Jirsa [11], on a 2/3 scale specimen of a RC frame with lightly reinforced columns and deep spandrel beams strengthened by an external bracing system are among the first studies in the U.S. The test specimen generally behaved in a very satisfactory manner. However, premature failure of some welds was observed which underscored the importance of careful detailing of connections and assurance of weld quality in such work. Also, failure of columns after buckling of braces showed that the vertical steel elements that are attached to the RC columns need to be designed for dual action -- i.e., as part of the truss action with the braces and as part of the RC frame in flexure after buckling and yielding of the braces.

Following the above mentioned studies, Lee and Goel [12] reported their findings from testing of a 2/3 scale specimen of a two story "weak" RC frame. The steel bracing system, consisting of ductile tubular braces in an inverted V-pattern and horizontal and vertical steel elements (collectors), was added to the RC frame after it was severely damaged during two earlier tests (Fig. 8). The upgraded frame showed excellent hysteretic behavior under a large number of cyclic deformations greater than 3% of story drifts. The hysteretic loops of the strengthened frame were very "full" and stable with a dramatic increase in energy dissipation capacity (Fig.9, 10). As was expected, a significant portion of the added strength was provided by the bracing members. However, a similar contribution came from the vertical and horizontal elements of the steel truss system through added moment resistance after buckling and yielding of the bracing members (Fig. 11). This extra strength was not anticipated in the design of the bracing system. Thus, the vertical and horizontal elements of the steel bracing system served dual purpose, i.e., as truss members and as moment resisting members in combination with the RC frame members. This is particularly remarkable since the vertical steel members were simply wrapped around the RC columns without any shear connection. It should be noted that direct connection of horizontal steel elements of the bracing system may be necessary to some degree in order to transfer the inertia forces from the floors to the bracing system. It is however practically convenient not to have to connect the steel elements to the columns. This is quite commonly done in Mexican practice.

The above finding of Lee and Goel was further investigated in greater depth by Masri and Goel [13] in their study of strengthening older RC flat slab-column building structures. They used a representative one-third scale, two bay, two story model for experimental and analytical study. The dual role of the horizontal and vertical steel elements was considered in design and analysis. The horizontal steel elements were attached to the top and bottom of the slabs by epoxy-grouted bolts, whereas, the vertical steel elements were simply wrapped and tied by horizontal batten plates around the columns without even using grout in between. The test frame is shown in Fig. 12. The main study was supplemented by cyclic bending tests and analysis of a series of RC members with steel elements wrapped around as was done for the columns of the main specimen. The behavior of the control RC beam was greatly improved by this jacketing technique. The "semi-composite" action of the encasing steel was quantified and the results implemented in designing the jacketing of the columns of the two-story test specimen.

Three tests were performed on the strengthened frame where gravity and cyclic lateral loads were applied simultaneously. Sand bags were used on the flat slabs at both levels to simulate expected gravity loads during design level severe earthquake motions. In the first test, only the horizontal and vertical steel elements were added in one of the bays. The objective was to study the frame action of the added steel in composite action with the RC frame. In the second and third tests chevron braces were added to one bay at a time. The results from the three tests showed excellent performance of the strengthened frame (Figs. 13, 14, 15). Excellent "semi-composite" flexural action developed between the vertical steel elements and the core RC columns which were able to force their curvatures on the jacketing steel. Analytical models and procedures were developed and used for nonlinear analysis. The analytical models were subjected to loads similar to those applied in the tests. The experimental and analytical results compared very well. The conclusions and analytical procedures developed in the study can be used for practical design, analysis and construction of the strengthening system for similar structures.

Use of prestressed cables and tendons has been proposed by some investigators. These elements are easier to install and eliminate the problems associated with buckling of structural steel braces. Studies have been reported by Miranda and Bertero [14] and Pincheira and Jirsa [15]. In the author's opinion, this system can be quite effective where stiffness and strength enhancement are primary design goals. Energy dissipation of such schemes is most questionable.

3.3 Damping

A few studies have been reported on upgrading systems which employ energy dissipation through inelastic hysteretic behavior of specially designed devices. Such devices are often used in combination with bracing members which are designed not to yield or buckle before yielding of the devices. An example of this scheme was shown earlier in Fig. 7. Seki, Katsumata and Takeda [16] reported tests carried out on a two story steel frame. They also presented an analytical feasibility study for upgrading an example building by their proposed method. In its most simple form the device can be considered like a vertical shear link. This technique appears to have good potential for application in RC frames also. However, to the knowledge of this author no such study has been reported to date.

3.4 Connections

Connection between the steel strengthening system and the existing frame is perhaps the most important element in any strengthening scheme. Almost all researchers and practitioners agree that this is the most critical and challenging area on which the success of any strengthening scheme depends. However, very few studies have been reported to date on this topic. The connections between the steel elements and existing RC frame can be classified as direct and indirect. Direct type connections attach the steel elements by means of some kind of epoxied bolts or dowels (Fig. 5), or direct bond through adhesive action of epoxy materials. An example of indirect connection is attachment of prefabricated strengthening panels to the RC frame through dowels or shear studs attached to both systems. The space between the two is filled with proper mortar after the panels are set in position (Fig. 6). Indirect connections tend to be more detail intensive. However, they provide the flexibility of ease of placement and adjustment, and lesser welding work of steel elements on site.

Behavior of connections between the two systems is quite complex. An example is shown in Fig. 16, where the connectors between the RC frame and the steel system may be subjected to complex combination and distribution of shear, axial and flexural stresses. Katagiri et al. [17], and Yamamoto and Aoyama [18] reported some tests on mortar joints of indirect type under direct shear. Studies by Weiner and Jirsa [19], and Jimenez and Kreger [20] focused on parameters influencing the behavior of steel elements attached to concrete by epoxy-grouted bolts under simple shearing forces. A lot more work is needed in this area before a more complete understanding of connections is attained and rational practical design procedures can be formulated.

4.0 CONCLUDING REMARKS

A brief review of state-of-practice and research on the subject of upgrading of older RC building frames by using steel elements is presented in this paper. The coverage is not claimed to be complete by any means. Only the most commonly used techniques known to the author have been mentioned. The subject is too broad and the behavior of the strengthened systems and their components is very complex. The research work carried out and reported to date has provided some understanding of the behavior and possible problem areas. The studies have been somewhat limited in scope and complexity. In view of its importance to public safety and possible life and property losses during future major earthquakes, the subject warrants a major comprehensive, and well planned research program with involvement of perhaps international resources and cooperation of researchers and practitioners before a more complete understanding is obtained and rational and effective design analysis procedures can be formulated.

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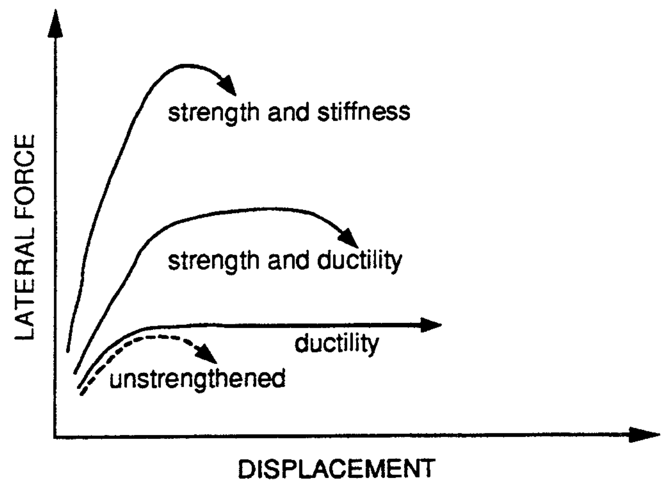

Fig. 1 Concepts of Seismic Strengthening.

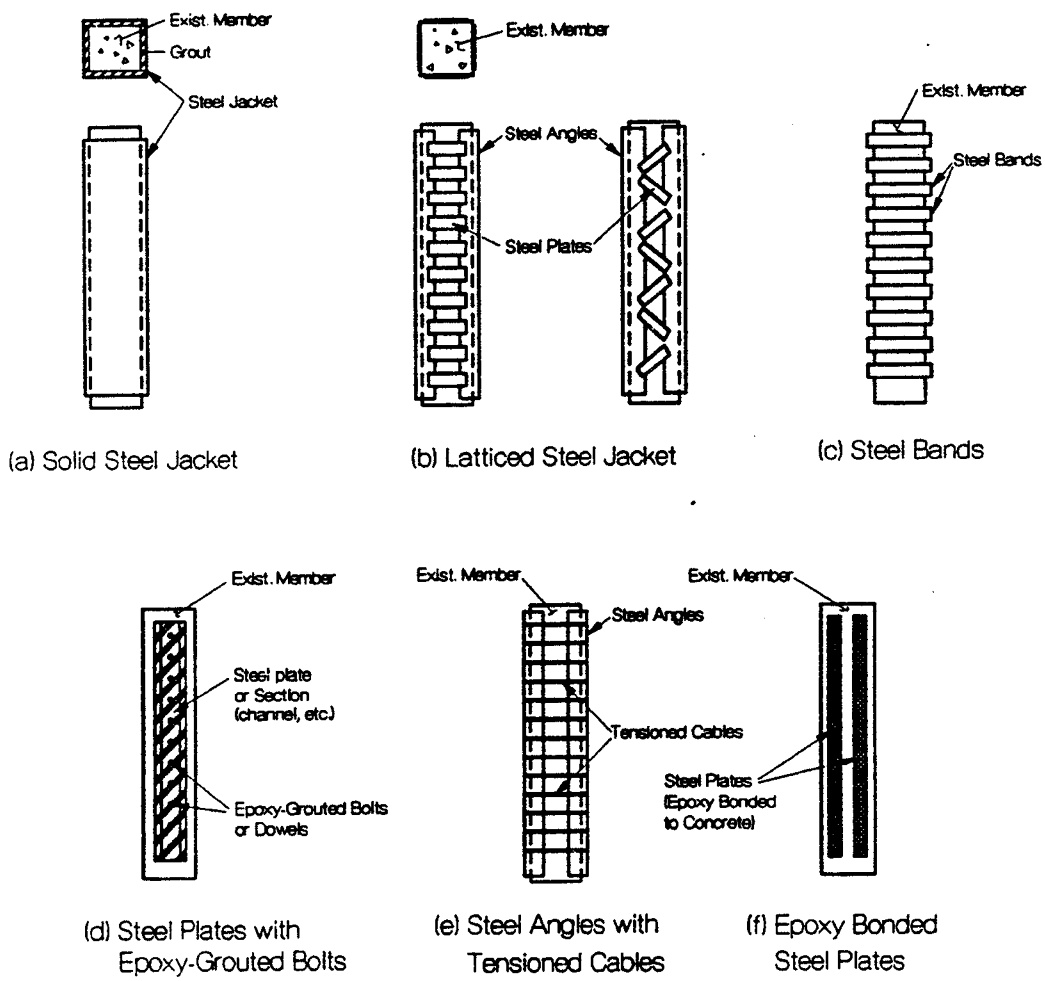

Fig. 2 Examples of Strengthening Techniques Using External Steel Jackets and Plates (From Ref. 2).

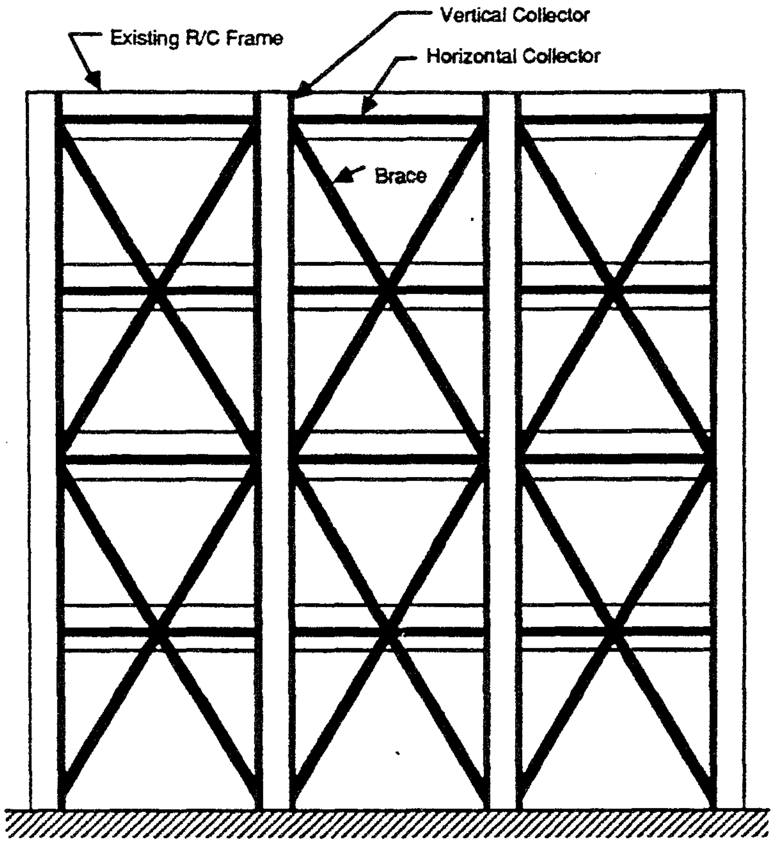

Fig. 3 Example of Steel Bracing System attached to RC Frame Piece-by-Piece.

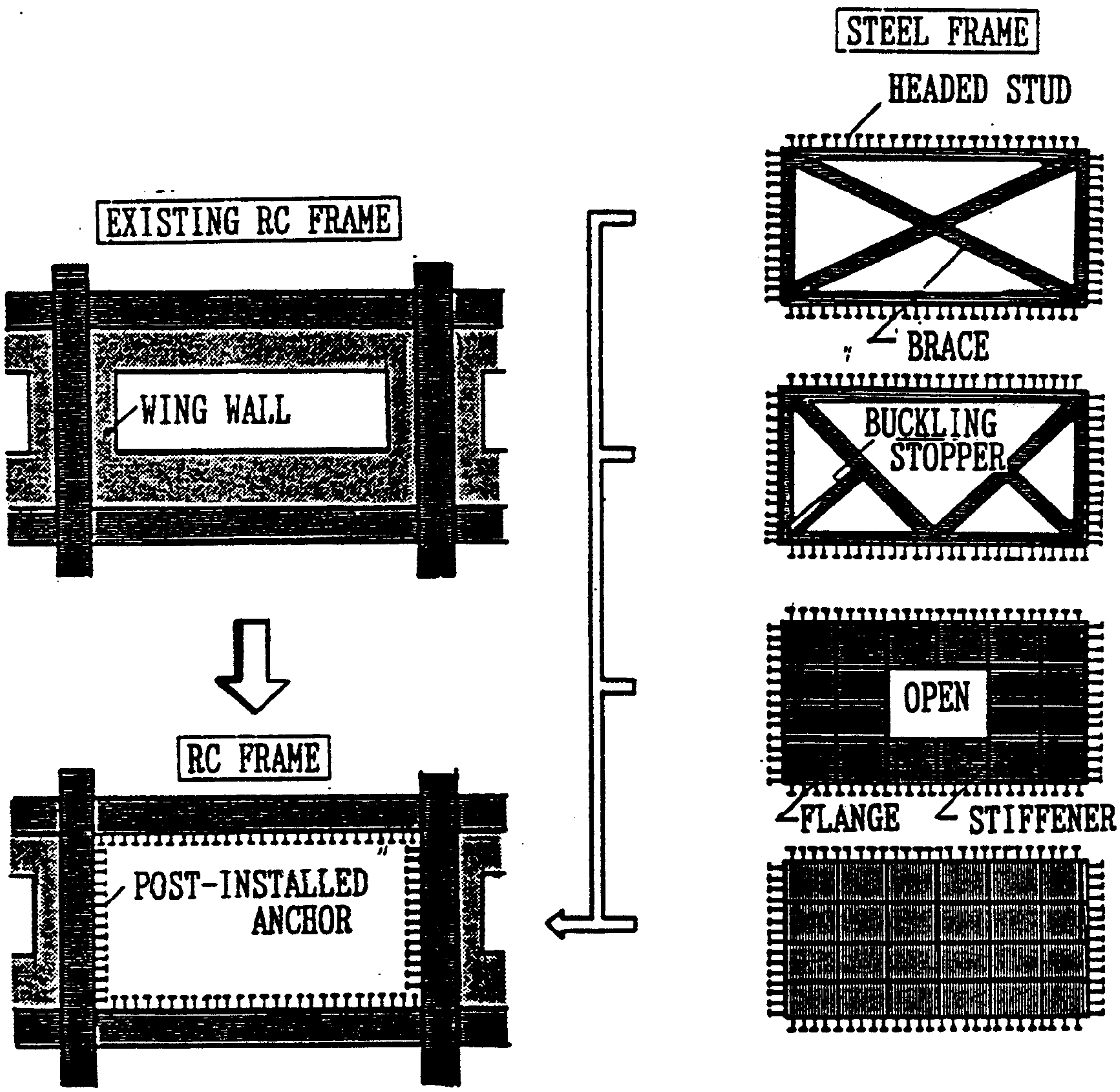

Fig. 4 Example of Pre-fabricated Bracing or stiffened Panel attached to RC Frame by Indirect Connection (From Ref. 8).

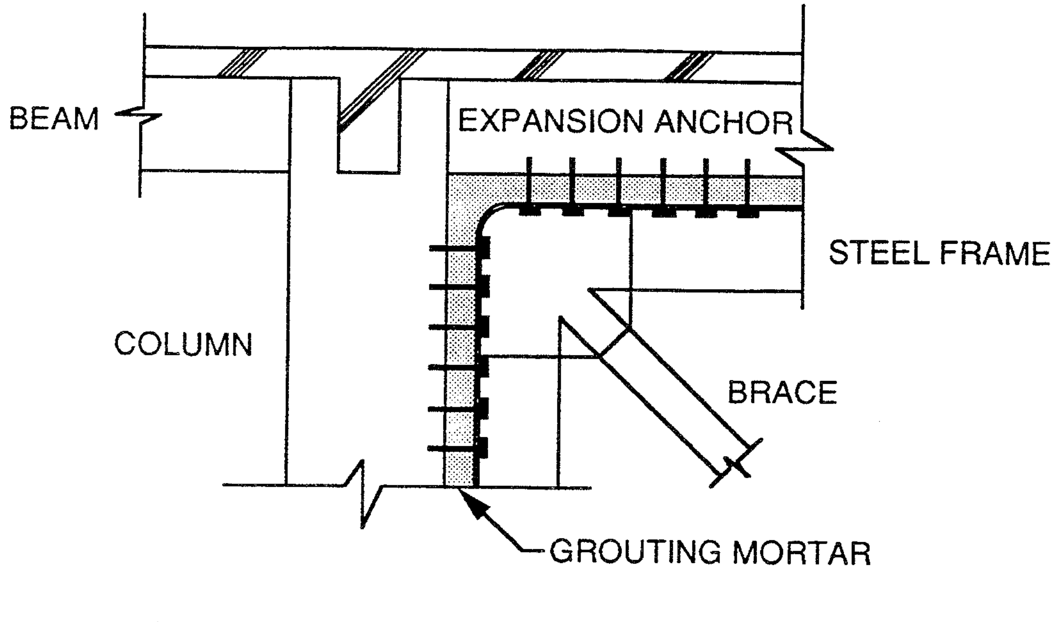

Fig. 5 Example of Direct Connection.

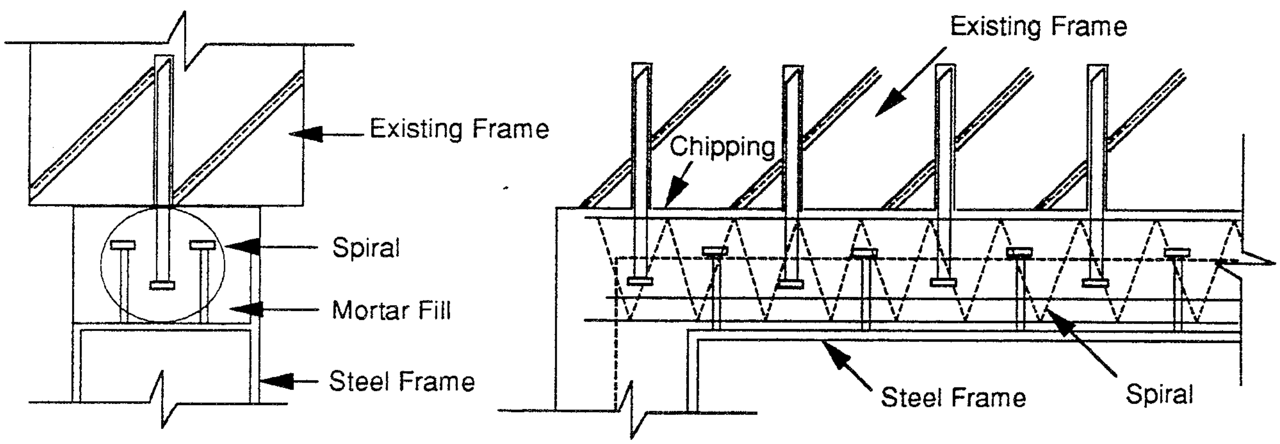

Fig. 6 Example of Indirect Connection.

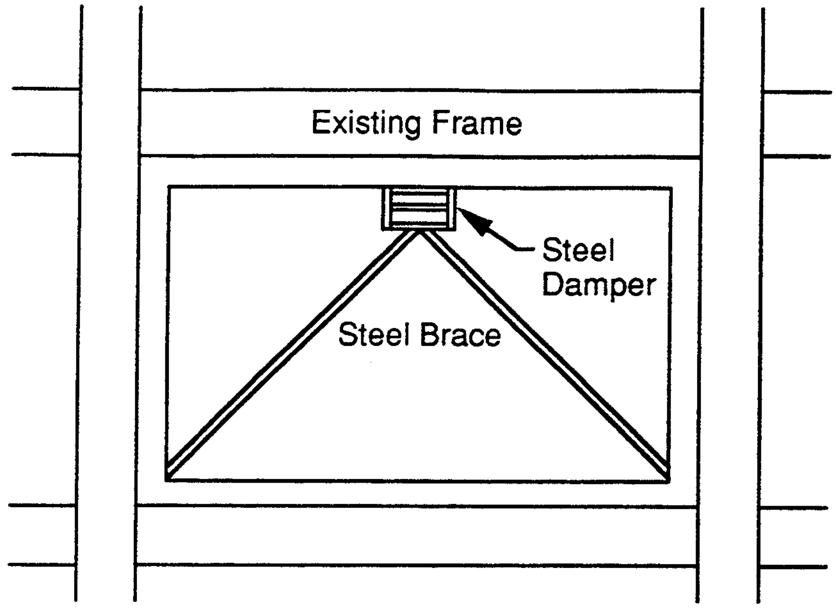

Fig. 7 Example of Strengthening by Damping Device.

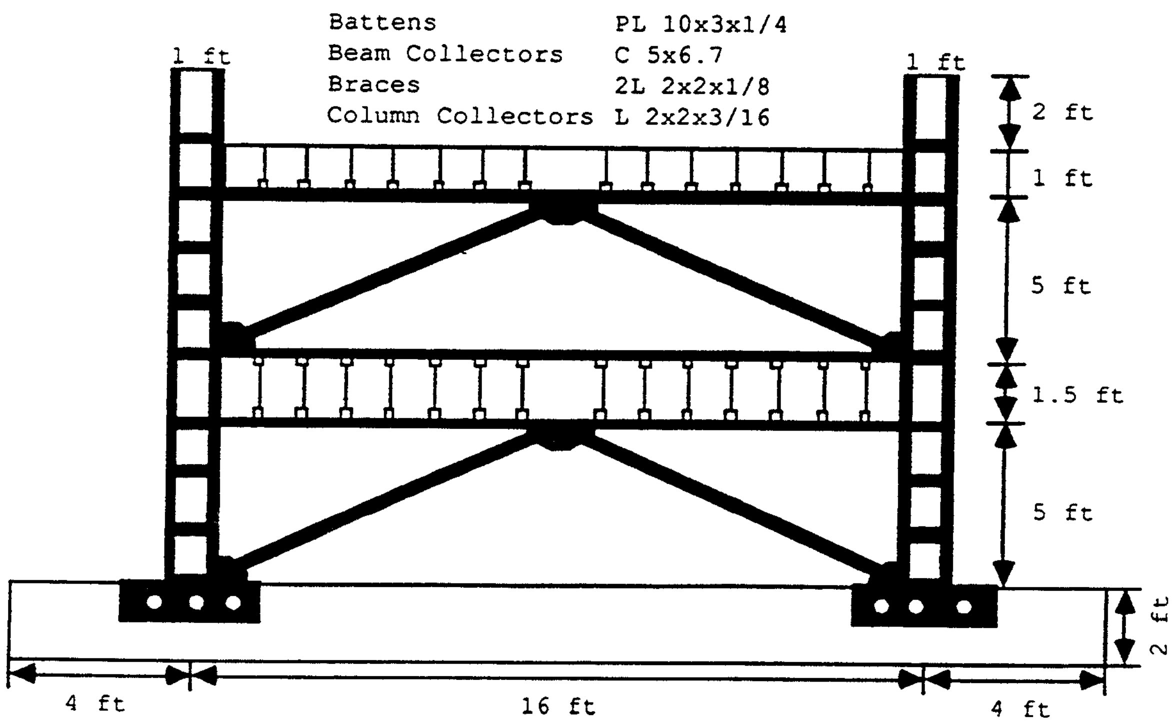

Fig. 8 Details of Steel Bracing System.

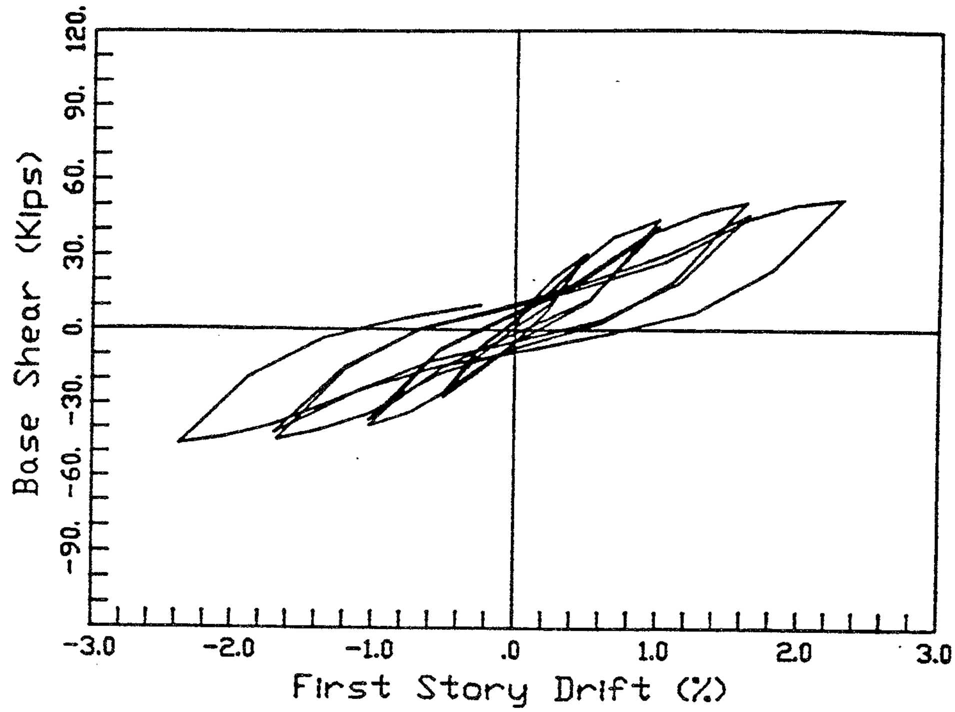

Fig. 9 Hysteretic Loops of Bare R.C. Frame.

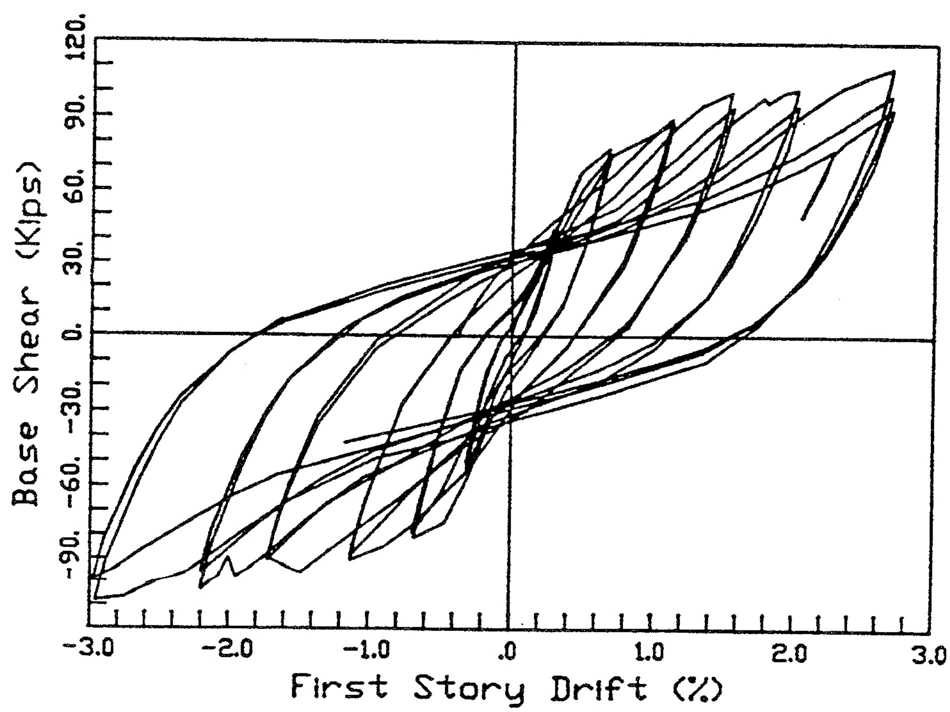

Fig. 10 Hysteretic Loops of Strengthened Frame.

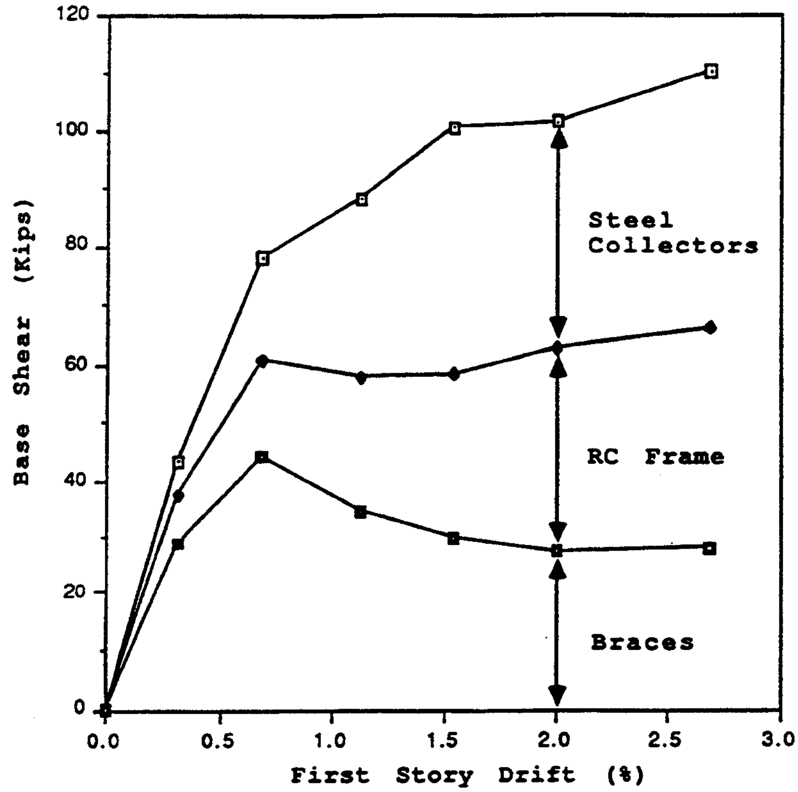

Fig. 11 Contribution of Each Component to Base Shear.

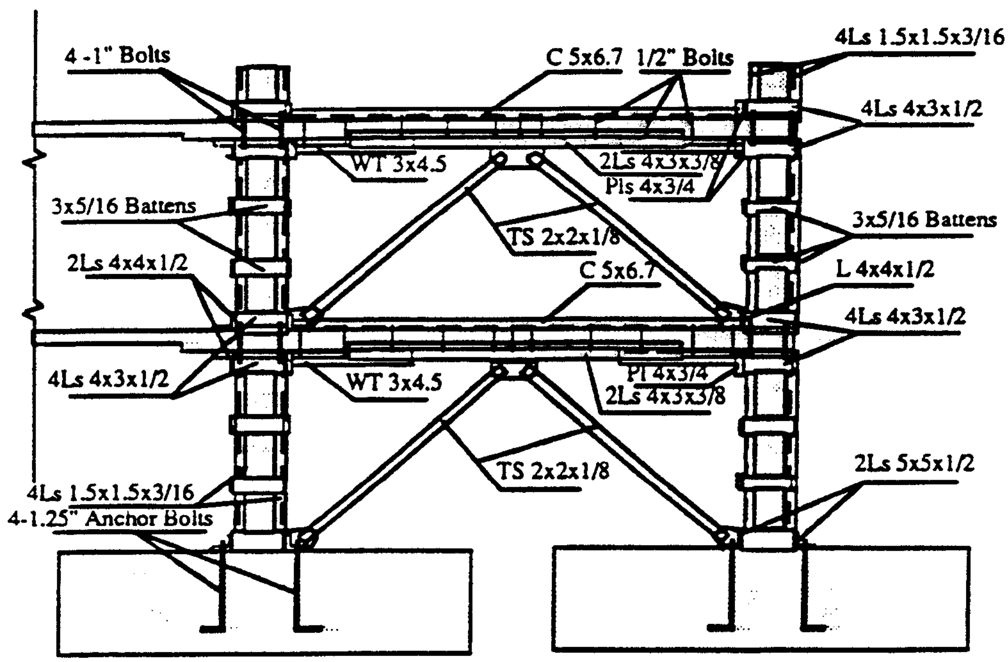

Fig. 12 Layout of the Frame with Bracing in One Bay.

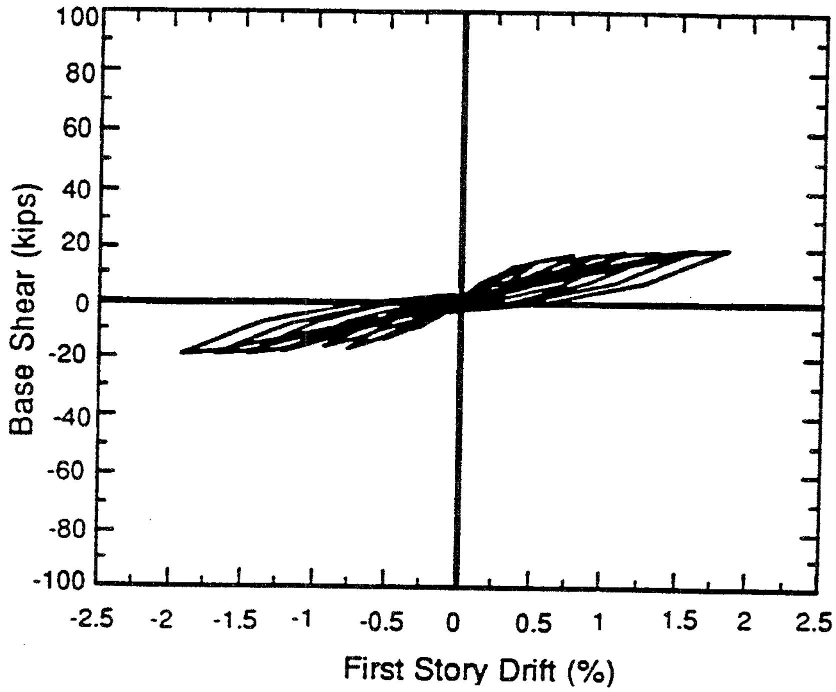

Fig. 13 Hysteretic Loops of the RC Frame.

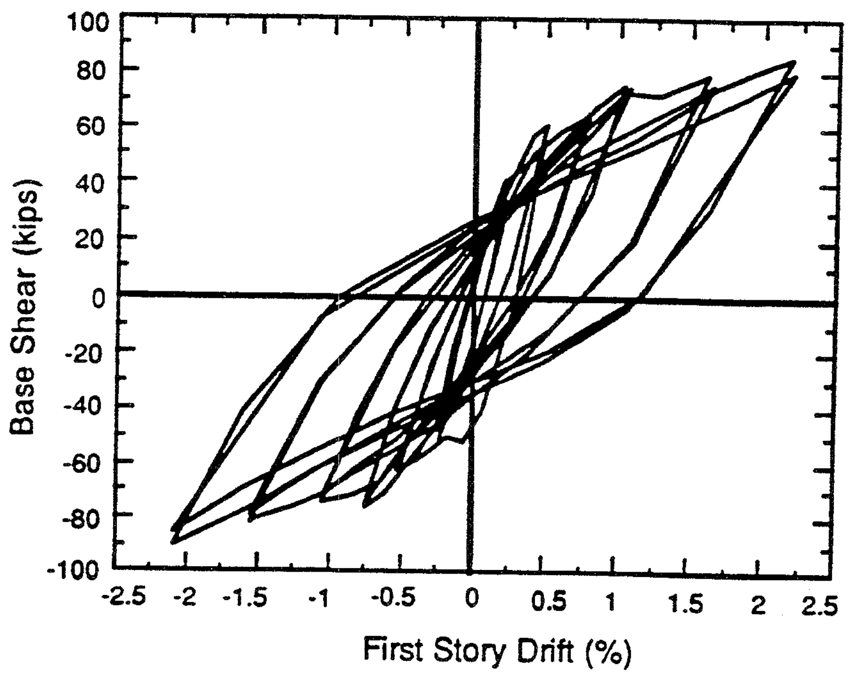

Fig. 14 Hysteretic Loops of the Braced Frame.

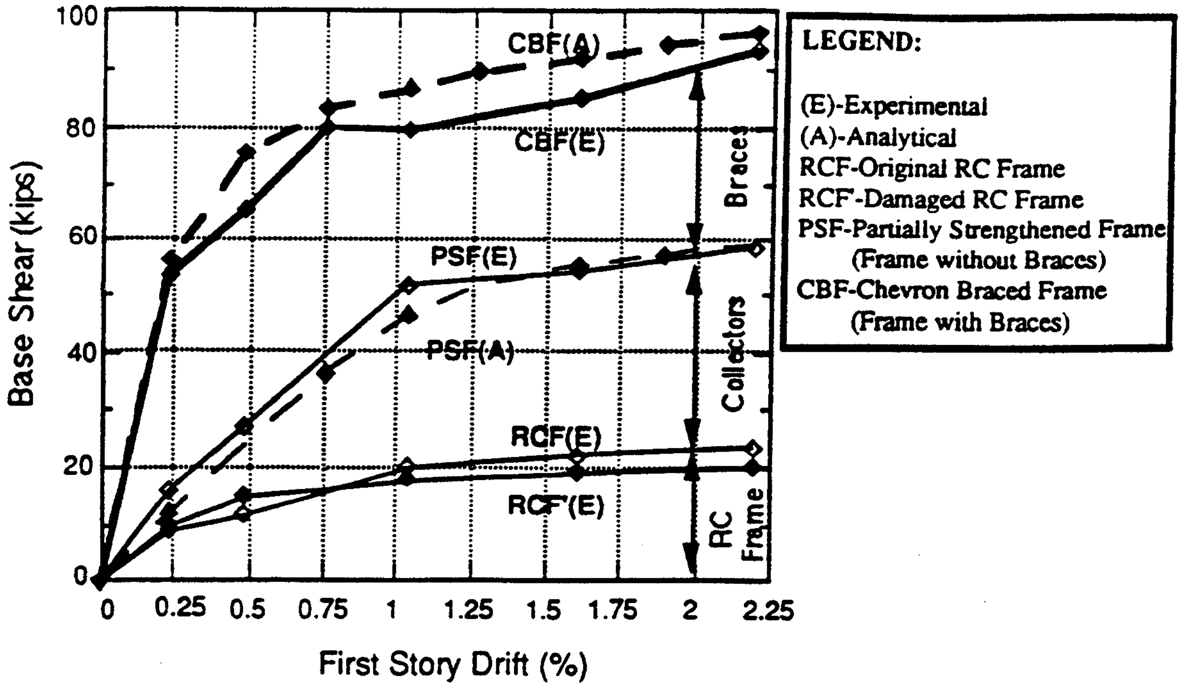

Fig. 15 Components of Lateral Strength.

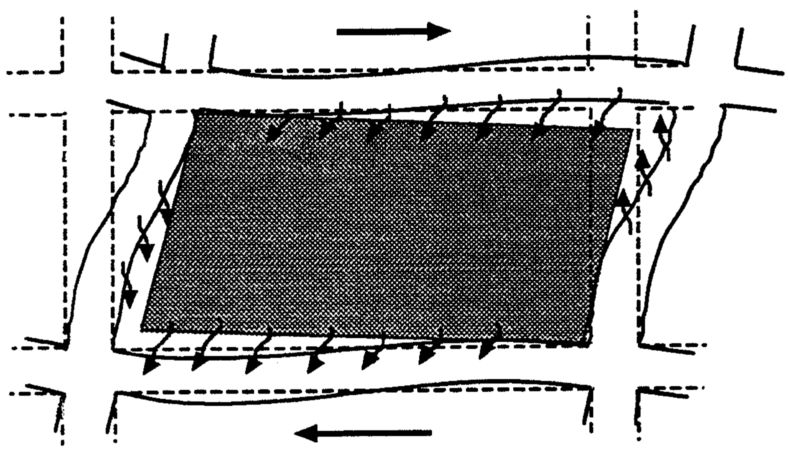

Fig. 16 Relative Deformation between RC Frame and Steel Panel.

USE OF STEEL ELEMENTS IN REHABILITATION OF RC FRAMES

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ABSTRACT

Structural systems constructed before recent code revisions (last 20 years) were adopted may have inadequate lateral capacity and ductility, poor anchorage details, inadequate column shear capacity, and/or poor anchorage of reinforcement. A series of projects conducted recently at the University of Texas have been aimed at improving the performance of existing moment-resisting reinforced concrete frames in zones of moderate to high seismicity using steel elements. A brief overview of the test program and of the implications of the research for design are presented.

1.0 BACKGROUND

Recent earthquakes have alerted the earthquake engineering profession to the hazards posed by existing reinforced concrete frame structures. Many frame systems were constructed in the US using design requirements that were developed before the 1971 San Fernando earthquake or are in regions of the United States where design for seismic effects has not had a high priority or has not even been considered. In such cases, the frames may be quite satisfactory for gravity loads but are lightly reinforced for earthquake effects. Although the San Fernando event led to substantial changes in seismic design requirements, especially lateral force levels and detailing requirements for transverse reinforcement and for continuity of reinforcement, a large number of structures constructed earlier remain in use.

A typical detail in a frame structure designed primarily to carry gravity and wind forces is shown in Fig. 1. "Weak links" in the structure include low column shear capacity, little or no positive moment capacity in the beams at the face of the column, inadequate moment capacity at the bottom of the columns where splice lengths are insufficient to develop yield, and insufficient confinement of the beam-column joints. Under moderate to severe ground motions, the structure would likely exhibit large lateral deformations and shear failures in the columns and joints could be expected. Rehabilitation techniques studied include encasing elements with steel or reinforced concrete jackets to correct shear, anchorage or ductility problems, adding steel elements to correct anchorage deficiencies, and adding infills or bracing to create new lateral load systems. Techniques studied for correcting such deficiencies will be briefly described.

2.0 IMPROVEMENTS IN MEMBER PERFORMANCE

2.1 Column Deficiencies

A key problem is the lack of shear capacity and/or ductility at critical sections along the column. Aboutaha [1] and Estrada [2] have studied the use of steel jackets to improve shear capacity of columns. Figure 2 shows the test procedure used by Aboutaha. The results of several tests are shown in Fig. 3. The results indicate that complete or partial encasement of the section was very effective provided that careful attention is given to details of the connection between the partial jackets and the existing column. Partial jackets were also studied by Estrada for a case where the architectural features of the building precluded the removal of the curtain walls for construction of jackets around the perimeter of the column.

Another weak link in columns is the lack of continuity of longitudinal reinforcement. Columns in older buildings often contain splices designed for compression only. In cases where the column may be subjected to large moments or subjected to tension (especially when infill walls are added and the column serves as a boundary element for the wall), the splices are likely to fail in tension. Aboutaha [3] has shown that steel jackets are very effective in confining the reinforcement and permitting the splice to develop large tensile strains. Figure 4 show several tests and the results using steel jackets to confine the column region where the splice is located. Valluvan [4] has also studied the case of use of simpler confining techniques for improving the tensile capacity of splices. Figure 5 shows the results of tests with confinement provided with angles and straps and Fig. 6 shows results with external reinforcing bar ties. It is clear that any external reinforcement must be grouted so that the external reinforcement is mobilized immediately.

2.2 Beam Deficiencies

As indicated in Fig. 1, a common problem in many frame systems is the lack of continuity (anchorage or splice length) in the bottom reinforcement of beams. Estrada [2] tested a specimen in which steel plates were attached to the bottom surface of the beams on opposite sides of the column and the plates were connected through the column with a large steel bolt as shown in Fig. 7. The plates were attached to the beam with epoxy-grouted dowels. A layer of grout was placed between the beam surface and the steel plates. Under cyclic loads, the strengthened beams performed very well as indicated in Fig. 8. The excellent performance was obtained because the plates reached yield and all the grouted bolts were able to develop their full capacity--largely because they deformed in a ductile manner in flexure rather than fail in direct shear. The grout layer between the plates and the bottom of the beam permitted the bolt to deform.

2.3 Grouted Bolt Connections

Jimenez [5] continued work begun by Weiner [6] and studied in detail the parameters which influence the performance of steel plates attached to concrete with epoxy-grouted bolts. An interesting aspect of this research is that the use of epoxy to fill the gap between the bolt and the hole in the steel element significantly improved shear transfer between the steel element and the concrete members (Figs. 9, 10). The tests also confirmed the observations in Estrada's test that a grout layer between the steel element and the concrete surface improved the ductility of the connection and permitted forces to redistribute before the highly stressed bolt failed in shear or fractured.

3.0 USE OF ALTERNATE LATERAL FORCE-CARRYING ELEMENTS

In some cases, difficulties associated with the correction of member deficiencies preclude the incorporation of existing frame members or frames into the primary lateral load resisting system. The use of alternate lateral load systems may provide the necessary strength and stiffness to control lateral deformations and to prevent a brittle failure in the existing frame elements. In some cases, it may be necessary to combine both approaches by providing an alternate lateral load system but improving the ductility of critical regions of the frame by jacketing with steel elements. Concrete infill walls or steel bracing elements are the most common approaches. The use of steel bracing in the form of structural steel braces or steel tendons will be discussed.

3.1 Structural Steel Braces

Bush, et al [7,8] tested a 2/3 scale reinforced concrete frame representing a typical 1950's design used in California. The prototype structure was designed with all the lateral resistance in the perimeter frames. The perimeter frames consisted of deep spandrel beams with small columns that were very lightly reinforced for shear. Many previous earthquakes have shown the poor performance of such frames with weak "captive" columns. Braces consisted of modified wide-flange sections of A36 steel. In addition to the steel braces, steel collectors were provided at the floor levels and along the columns to collect shear forces from the floors and to provide vertical reactions to the braces at the column which protruded from the face of the spandrel beams as shown in Fig. 11. All collector elements were attached to the concrete with grouted anchor bolts similar to those tested by Jimenez and Weiner. There were some difficulties encountered in attaching the collector elements because the holes often intersected column or beam reinforcement which could not be located accurately prior to drilling. Welding also posed some problems because of the small sections and the lack of space at the intersections of braces and collector elements.

During testing some welds failed prematurely and had to be rewelded. These difficulties emphasize one important aspect of steel bracing--the need for careful layout of all weld details and strict control of all welding procedures. In general, the steel bracing system performed well. The response was linear up to levels about twice the design code forces. When the braces finally buckled the load was shifted suddenly to the columns and they failed in shear.

3.2 Steel Tendon Braces

To avoid some of the problems associated with buckling of steel braces and failure of connections, a number of investigators have proposed the use of cable braces. The braces are relatively easy to install and may be prestressed to control deformation and to avoid the braces becoming slack under loads in the opposite direction. These systems have been used in Mexico City after the 1985 earthquake and have been studied by Bertero and Miranda [9]. Pincheira [10] has conducted analytical studies of these systems to determine the influence of prestress level and tendon brace details on performance under large deformation reversals. Algorithms for the response of critical elements in the systems are based on experimental results. Using a number of ground motion records, the performance of structures rehabilitated with different techniques has been estimated. Figure 12 shows the results of one study of a twelve story structure subjected to different ground motions and with two brace details. The results indicate that any rehabilitation solution must be examined considering the nature of the ground motions at the site and the characteristics of the structure including the "weak links" in the structure and the dynamic properties of the structure.

4.0 CLOSING COMMENTS

4.1 Organization of Research

The repair and strengthening of reinforced concrete frame systems involves a complex interaction of new and existing elements. A great deal of information is needed regarding the response of both existing and rehabilitated elements and structures in order to develop a strategy for meeting the building owner's performance objectives. The goal of the studies described here has been to provide information for improving the reliability of assessments of existing structures and the designs of rehabilitation schemes. An important aspect of the studies at the University of Texas has been the continuous and direct involvement of design engineers in the planning of test programs, the design of test specimens and details of the rehabilitation schemes, and in the evaluation and reporting of results (8). The collaboration of research and practicing engineers is essential for producing results that will be quickly transferred from the laboratory to the field and especially to rehabilitation codes and guidelines.

4.2 Future Research Needs

The combinations of structural systems, materials, and earthquake force or recurrence levels makes it very difficult to cover the wide variety of systems with a few tests. It has been possible to isolate some common deficiencies and to study various rehabilitation approaches for correcting these problems. However, the major problem confronting designers continues to be the lack of procedures for evaluating the performance of both existing and rehabilitated structures. Attention will need to be given to experimental studies of complete systems or, at the least, large segments of such structures. Static cyclic tests, shake table studies, and field studies and instrumentation all need to be developed. Until a systematic program of work is developed to address the mitigation of hazards posed by existing inadequate structures, it will not be possible to provide the kind of design guidance and cost/benefit data needed to correct a problem of potentially devastating impact on the United States economy.

ACKNOWLEDGEMENTS

The author expresses his appreciation to the many students whose work is referenced here. Their hard work and dedication to the research program at the University of Texas has been instrumental in the success of the program. Without their involvement and that of Professors M. E. Kreger and M. D. Engelhardt, the research would not have been possible. The support of the National Science Foundation, Degenkolb Engineers, Englekirk & Sabol Inc., and A. C. Martin Associates for their role in the research described here is gratefully acknowledged.

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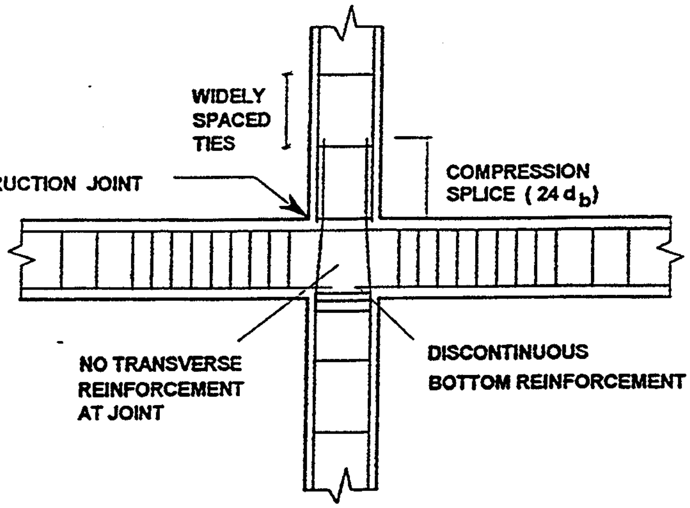


Fig. 1 Typical weak links in existing reinforced concrete moment resisting frames

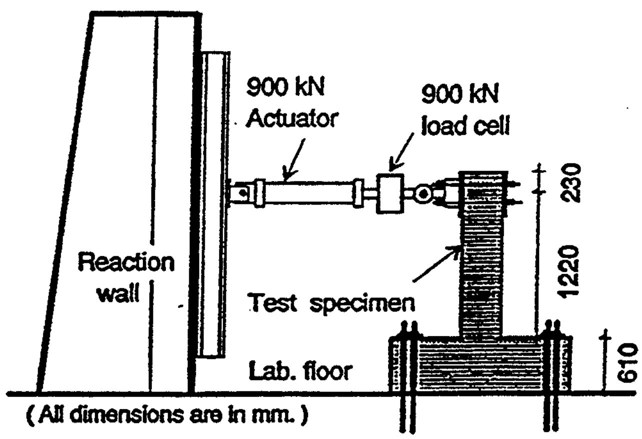


Fig. 2 Test setup for column shear and splice strengthening tests

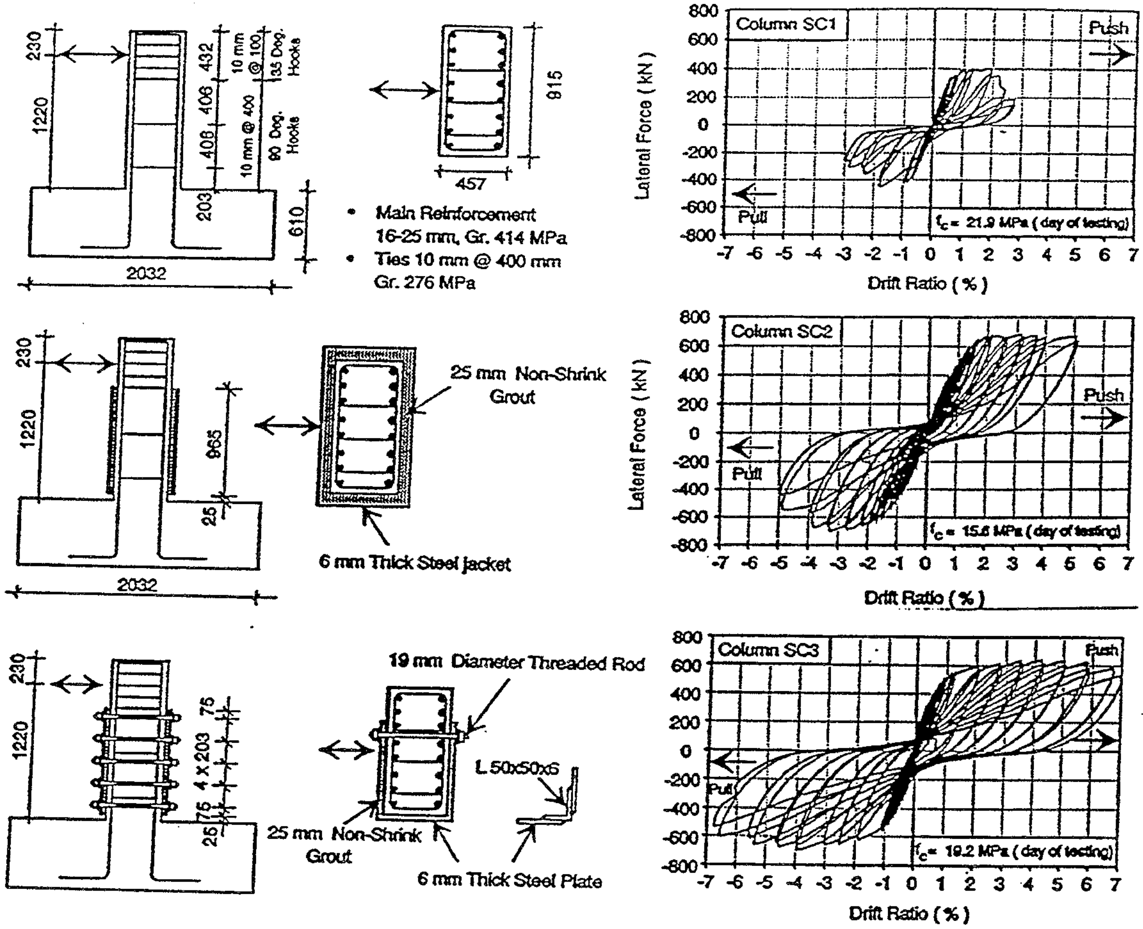


Fig. 3 Use of steel jackets to improve column shear capacity

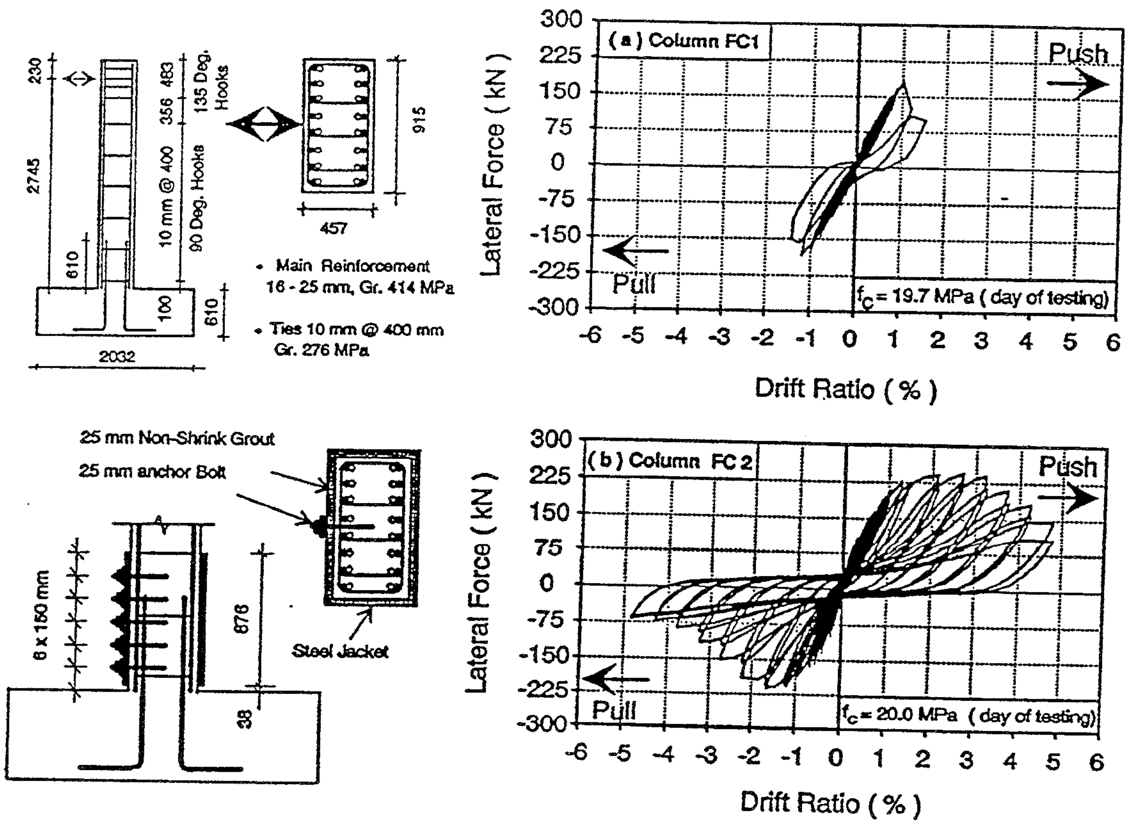


Fig. 4 Use of steel jackets to strengthen a column splice region

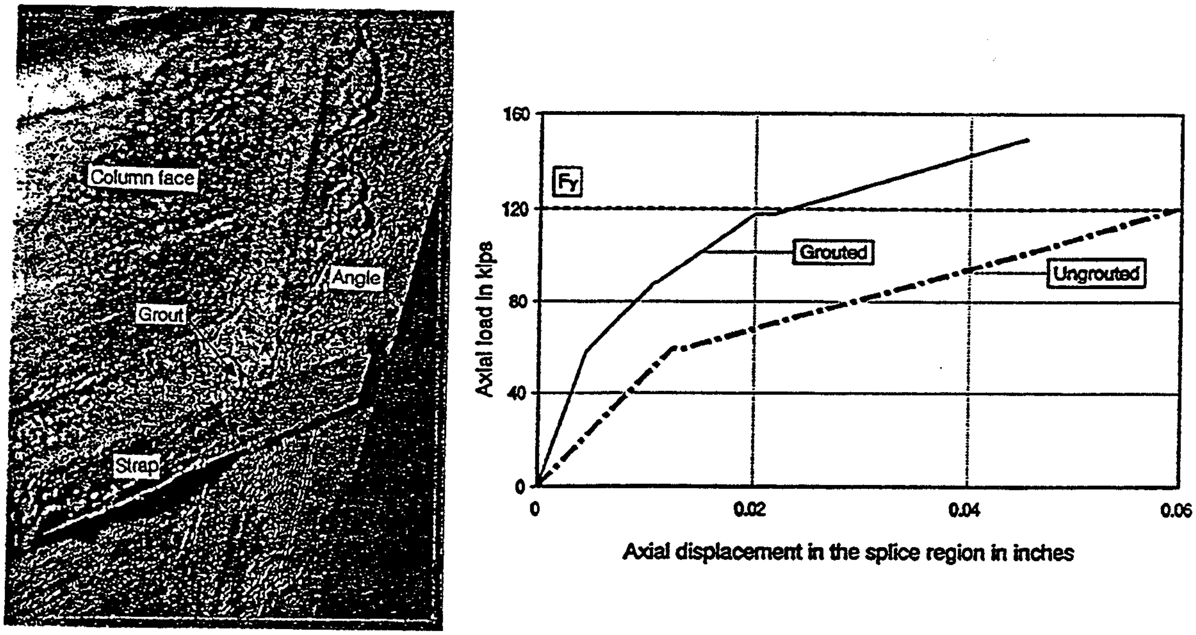


Fig. 5 Use of angles and straps to strengthen splice region

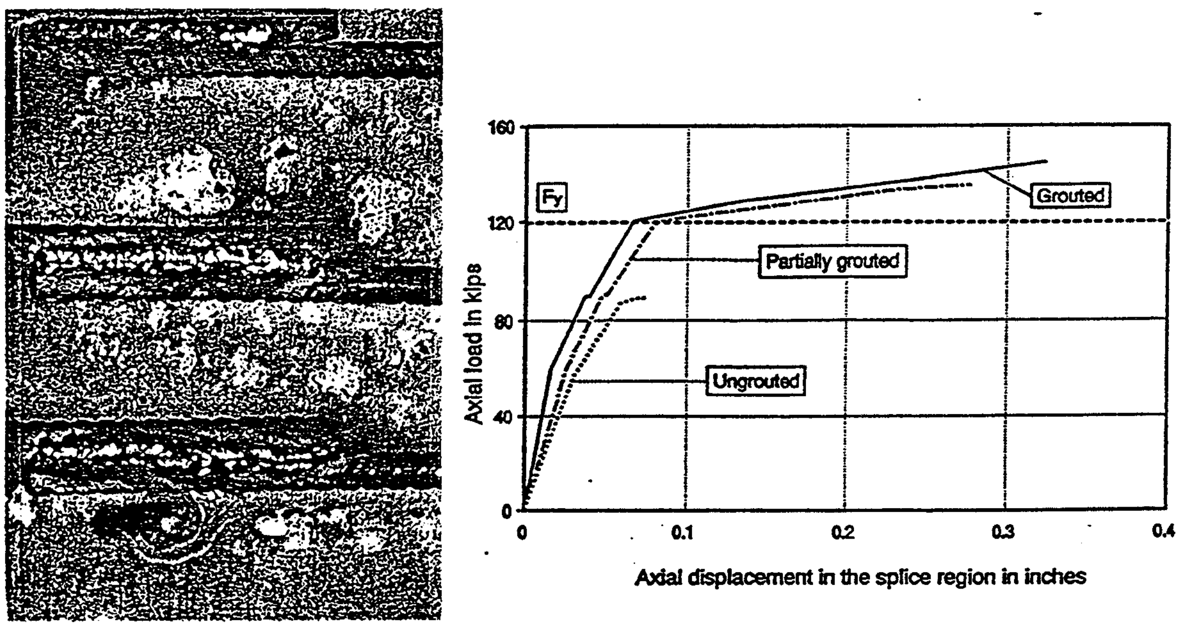


Fig. 6 Use of external ties to strengthen splice region

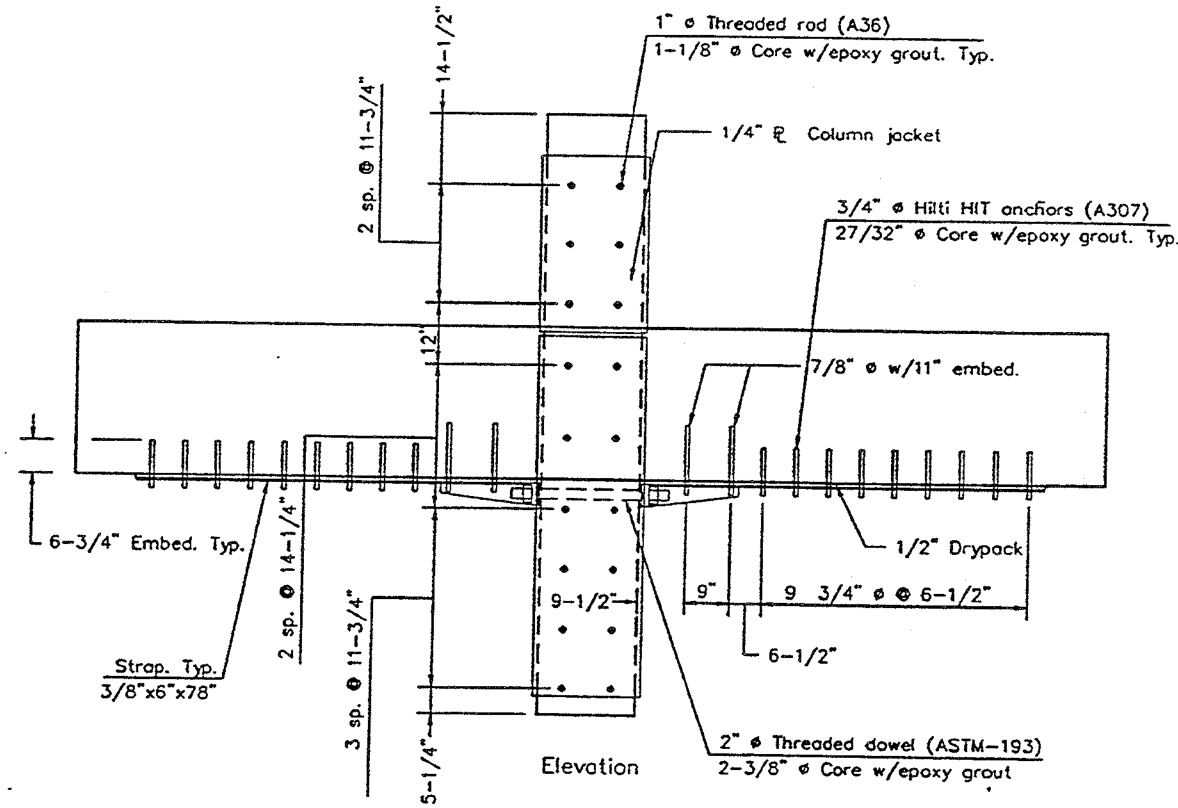


Fig. 7 Strengthening scheme for improving positive moment capacity in beam

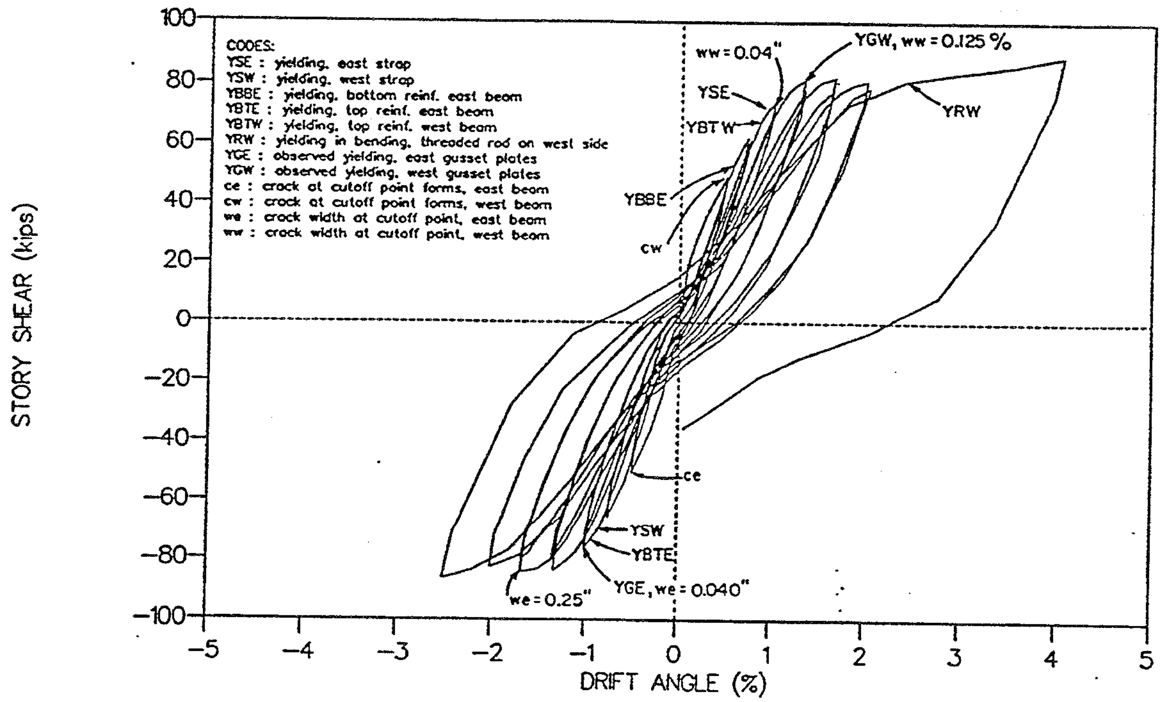


Fig. 8 Story shear vs drift angle for specimen with added positive moment capacity

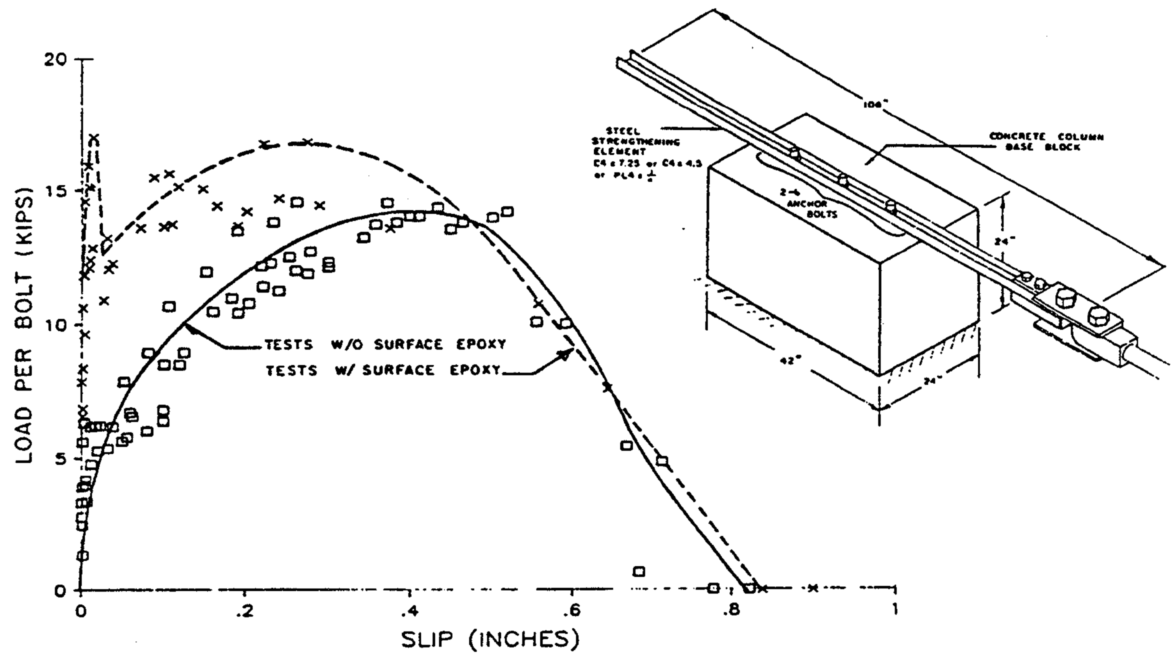
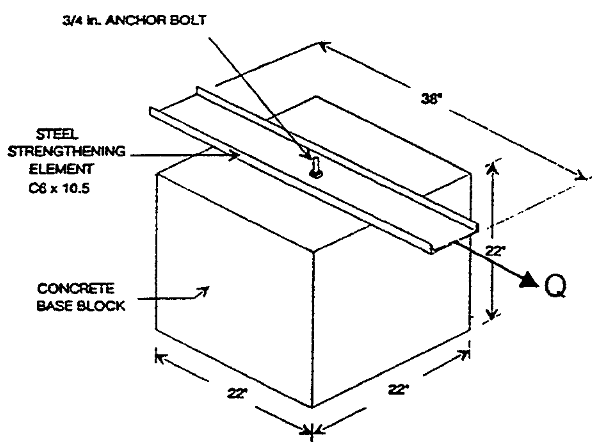
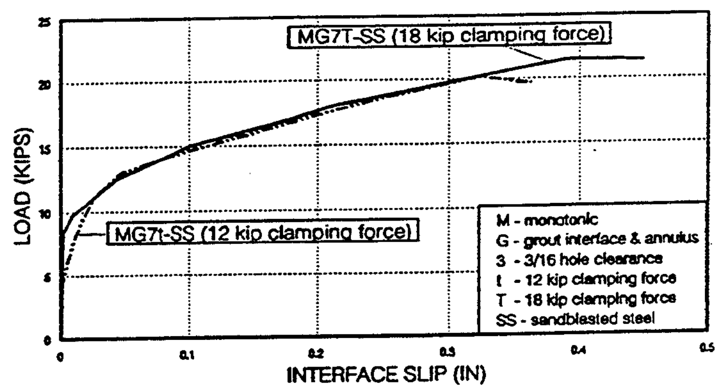
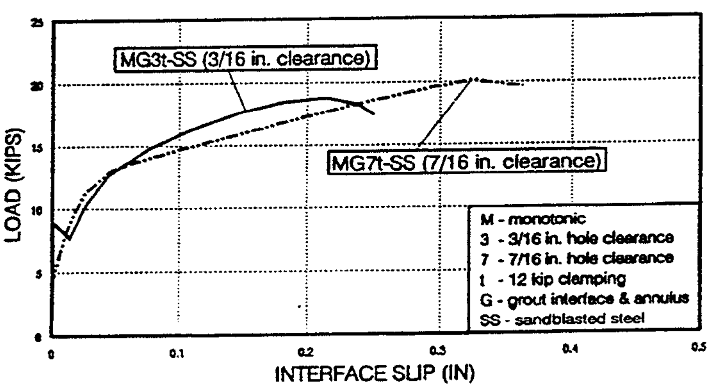


Fig. 9 Shear transfer with multiple bolt connections between steel and concrete

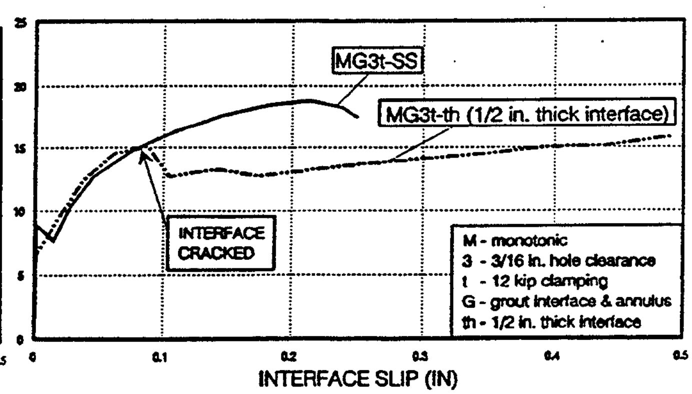




Clamping force

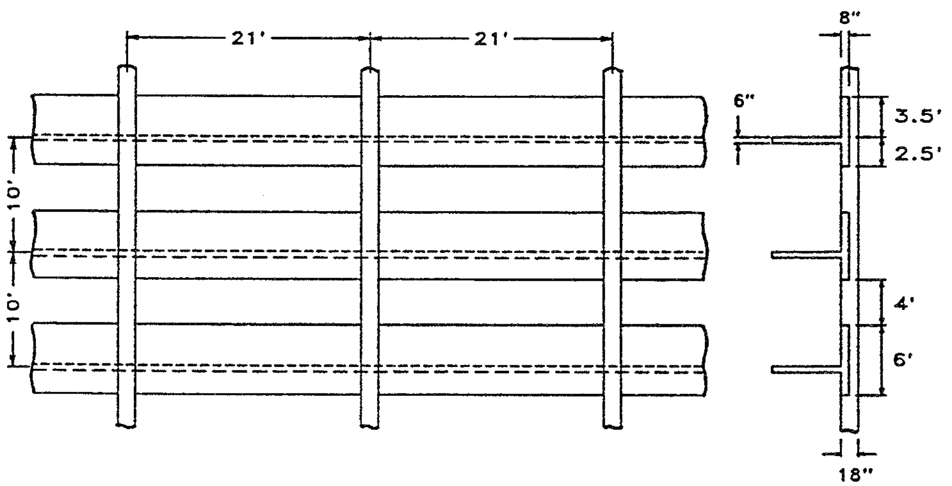


Hole clearance

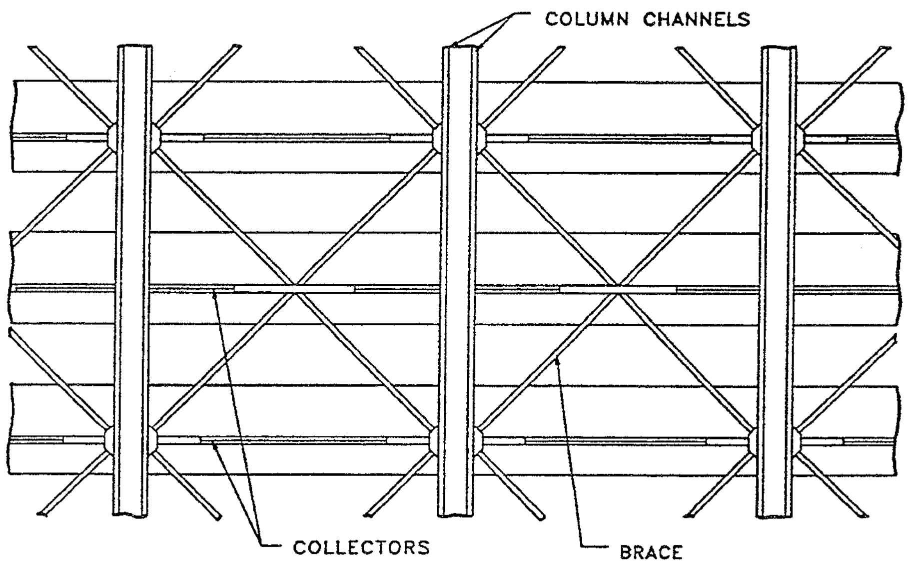


Interface thickness (with grout in annulus)

Fig. 10 Shear transfer with single bolt connections

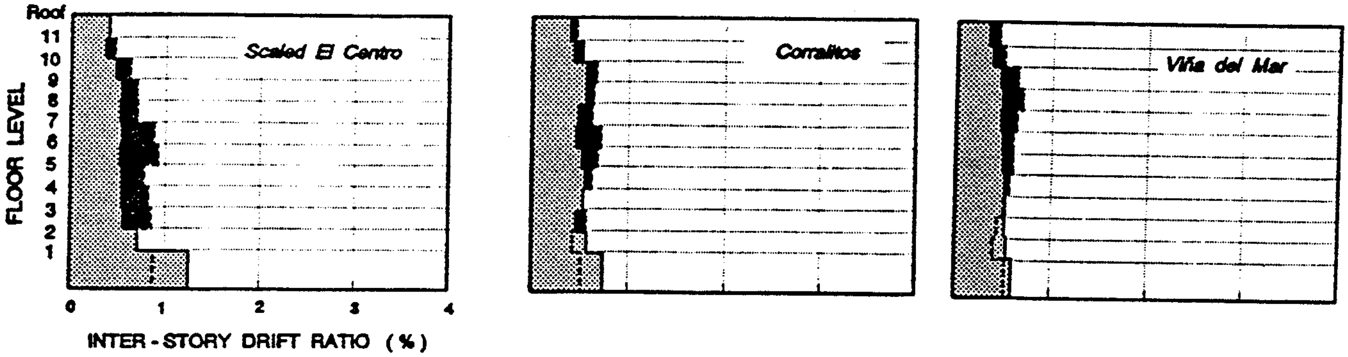


Prototype

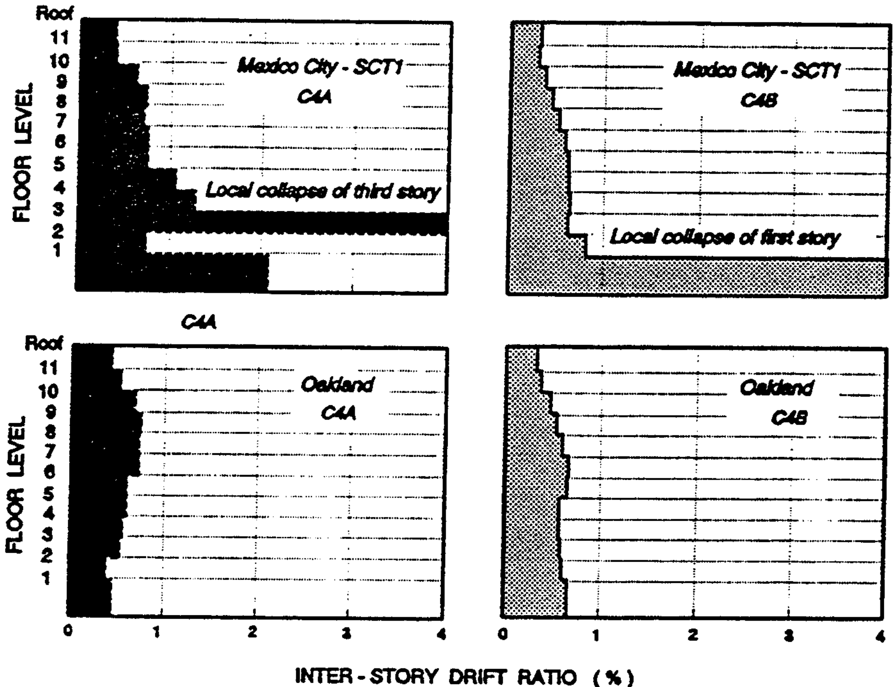


Brace layout

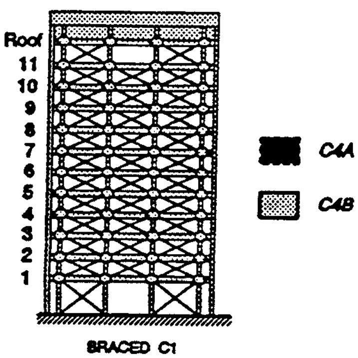
Fig. 11 Braced frame



Firm soil sites



Soft soil sites



Brace details

Story Level	Brace Size [Area (in^2)] C4A	Brace Size [Area (in^2)] C4B
3rd to Roof	2 - 1 3/8" ϕ rods [3.16]	3 - 1 3/8" ϕ rods [4.74]
2nd	3 - 1 3/8" ϕ rods [4.74]	3 - 1 3/8" ϕ rods [4.74]
1st	4 - 1 3/8" ϕ rods [6.32]	4 - 1 3/8" ϕ rods [6.32]

Fig. 12 Maximum inter-story drifts for 12-story building with cable braces

USE OF SUPPLEMENTAL DAMPING DEVICES FOR SEISMIC STRENGTHENING OF LIGHTLY REINFORCED CONCRETE FRAMES

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ABSTRACT

Devices which can add damping and strengthen structural systems, without changes to the existing components, were recently found to be extremely useful in retrofitting lightly reinforced frames and other structures. Some damping devices were developed purposely for building structures, however, many more were adapted from use in other industries such as the military and aerospace. Damping devices can provide also supplemental stiffening and strength to structures that lack such properties, in most cases without altering the existing components. The use of such devices does not require major disturbances or loss of functions of the structures during retrofit. However, a good understanding of the effects of the dampers on improving the structural capacity and reducing the seismic demand is essential to the design process. This paper surveys the state-of-the-art of seismic retrofit design, the role of supplemental dampers in such design and the currently available damping devices. The paper summarizes some validation studies using such devices for seismic retrofit and recommends further studies to address complex issues.

1.0 INTRODUCTION

Supplementary *damping systems* in reinforced concrete framing can enhance the damping (capability to dissipate energy), or increase the stiffness and strength to better control the deformations of the structure; thereby reducing the potential damage. Braces containing yielding metal elements, solid viscoelastic or fluid - viscous, or friction devices can be introduced into the lightly reinforced frames along with solid metal braces in a conventional retrofit scheme. The use of viscoelastic or fluid filled walls as infills for frames gained popularity in rehabilitation projects in Japan. All of the above techniques have the flexibility of providing either more damping or stiffness, or both, to better control the interaction with existing components and to reduce the seismic demands without modification of the existing structural components.

Newer techniques of external coating were suggested recently for the rehabilitation of lightly reinforced concrete structures. Among those techniques are the use of cementitious coatings, or more modern epoxy layers, reinforced with either glass, carbon, or steel fibers. These techniques were used in the rehabilitation of lightly reinforced concrete piers and walls.

More advanced techniques of semi-active or active braces, or mass dampers, can also be used for the rehabilitation of lightly reinforced concrete structures without modifications to the existing system. These devices process response information in real time and through force generators produce a substantial reduction of the seismic demands when needed, at the expense of additional energy. These techniques are new and basic questions still need to be answered.

Modern seismic design practice permits design using inertial forces lower than that expected for elastic response on the premise that inelastic action occurs and provides the structure with significant energy dissipation to survive severe events. The inelastic action is supposed to develop in specially detailed critical regions, usually near connections, that have ductile properties and the ability to dissipate energy without major deterioration. However, these practices are relatively new and many reinforced concrete structures were designed before such practices were in effect.

Recent major destructive earthquakes around the world (Kobe 1995, Northridge 1994, Loma Prieta 1989, Mexico City 1985) are reminders that many buildings and bridges are constructed with reinforced concrete framing systems, lightly reinforced with respect to the requirements of seismic damage control. In severe seismic zones many reinforced concrete framing systems were designed using deficient old seismic practices and are prone to damage. Some of the structures sustained damaged in previous earthquakes and are in need of repair and rehabilitation. Many others are undamaged, but need upgrading to withstand future events. Most structures in low or moderate seismic hazard areas were never designed for seismic resistance, but only for gravity loads, or for minor lateral wind loads. All these structures are defined herein as lightly reinforced concrete (LRC) structures.

Lightly reinforced framing systems lack either strength or toughness (ductility) due to insufficient seismic details. The task of rehabilitation is to provide suitable increase in strength capacity or reduce the seismic demand. Conventional techniques such as concrete jacketing, external reinforcing, etc., can improve weaker regions of beams, columns, or walls. More recently, large scale rehabilitation projects used additions of reinforced concrete walls or steel braces to increase the resistance of the weak framing systems. These conventional techniques, however, increase substantially the stiffness of elements and simultaneously increase the seismic demand. Most of the conventional techniques are invasive and disruptive to the function of the structures.

This paper provides the background to the expected contributions and improvements to structures rehabilitated using mechanical dampers or composite coatings. The state-of-the-art of energy dissipation devices is presented including examples of full scale applications to reinforced concrete (RC) structures and of small scale experimental studies. The state-of-the-art of modeling, analysis and design recommendations is also presented. Since most of techniques described are new, several questions and important issues that need further clarifications are also outlined in this paper.

2.0 EFFECTS OF REHABILITATION OF WEAK STRUCTURES

Lightly reinforced concrete frame structures are expected to develop inelastic deformations when stressed beyond their elastic limits, as shown in Figs. 1(a) and 1(b). An elastic-ideal-plastic model is used for simplicity of explanation. The effect of retrofit can be an enhancement of structural capacity by strengthening [increase of strength capacity, see Fig.1(a)] or by stiffening [adding stiffness to the entire system before and after yield, see Fig. 1(b)]. These enhancements are typical components of retrofit and combinations of such effects are expected. Another effect of retrofit can be a reduction of seismic demand which can be represented in terms of the composite spectrum of acceleration and displacement shown in Figs. 1(c) and (d). The effect of adding viscous damping to a structure illustrated in Fig. 1(c) is to "shrink" the composite spectrum reducing both accelerations and displacements by similar ratios. However, a similar effect to viscous damping is obtained by hysteretic energy dissipation of members in the structure or by adding sacrificial elements with hysteretic properties. (see Fig. 1(d)) In such cases, the reduction of accelerations and displacements it is not proportional as in the case with viscous damping.

The results of rehabilitation can be obtained by intersecting the capacity curves representing the structure with the demand spectra representing the seismic motion (see Fig.2). While simple strengthening [see Fig. 2(a)] may increase the response accelerations and the forces on the foundations, the inelastic response displacements are changed from their original, d_o, to the reduced value, d_r. Comparing the deformation demand ratios to the original ultimate deformation (d_r/d_u vs. d_o/d_u), a substantial reduction is obtained and with it, a reduction of the expected damage. A similar effect is obtained by stiffening the structure [see Fig. 2(b)]. The result of damping increase, illustrated in Fig. 2(c) and (d), indicates reductions of deformations without a reduction of accelerations, if strong inelastic deformations are expected. A substantial increase in damping may lead to a reduction of accelerations, if the structure is retrofitted to respond only elastically.

While structure rehabilitation through capacity enhancement or reduction of seismic demand reduces the inelastic deformations, the two techniques differ in their influence on the acceleration response. The strengthened or stiffened structure may produce undesired increases of accelerations, while increased damping may leave them unaltered. It should be noted that, in the actual retrofit implementations using "damping" devices, the structure capacity is enhanced with a concurrent reduction in the seismic demand as illustrated in Fig. 2.

Damping systems differ largely in their construction and in the manner in which they are connected to the structures. The efficacy of each system may depend on the structural system and on the specific local seismic demand. The most desirable goal in rehabilitation, however, is to increase damping alone without stiffening or strengthening. This is attained by the addition of supplemental damping, where feasible.

Another less obvious effect of added damping is the redistribution of the energy dissipation within the structure. The total energy input during a seismic event (Uang, 1988) is distributed between the kinetic energy, the recoverable elastic strain energy, the irrecoverable energy dissipated through inelastic hysteretic deformations and the energy dissipated by damping. The hysteretic energy dissipation throughout the structure is associated with loss of both strength and deformation capacity, d_u. However, reductions of deformation capacity below the deformation demand may lead to catastrophic collapse. The increased energy dissipation obtained by damping substantially reduces the demand for energy dissipation through hysteretic inelastic deformations which results in reduced damage potential.

3.0 DAMPING AND ENERGY DISSIPATING SYSTEMS

Supplemental damping systems are mechanical devices that can be incorporated in a frame structure to dissipate energy at discrete locations throughout the structure. These devices include either one of the yielding of mild steel, sliding friction, motion of pistons within fluids, orificing of fluid or viscoelastic action of elastomeric materials. A summary of the damping devices available for retrofit are listed in Table 1.

3.1 Yielding Steel Elements

The reliable yielding properties of mild steel have been explored in a variety of ways for improving the seismic performance of structures. The eccentrically braced frame (Roeder 1978) represents a widely accepted concept. Energy dissipation is primarily concentrated at specifically detailed shear links of eccentrically-braced steel frames, which are likely to suffer only localized damage in severe earthquakes. A number of mild steel devices have been developed in New Zealand (Tyler 1978, Skinner 1980) and some were widely used in seismic isolation applications in Japan (Kelly 1988).

Tyler (1985) described tests on a steel element fabricated from round steel bar and incorporated in the bracing of frames. Figure 3 shows details of a similar bracing system which was installed in a building in New Zealand. Energy is dissipated by inelastic deformation of the rectangular steel frame in the diagonal direction of the tension brace.

Another element, called "Added Damping And Stiffness" or ADAS device studied by Bergman and Goel (1987), Whittaker (1989) and Hanson (1993), consists of multiple X-shaped steel plates shown in Fig. 4 and installed as illustrated. The shape of the device is such that yielding occurs over the entire length of the device. Shake table tests of a three-story steel model structure by Whittaker (1989) demonstrated that the ADAS elements improved the behavior of the moment-resisting frame to which they were installed by a) increasing its stiffness, b)increasing its strength and c) increasing its ability to dissipate energy. Ratios of recorded interstory drifts in the structure with ADAS elements to interstory drifts in the moment-resisting frame without ADAS elements were typically in the range of 0.3 to 0.7. This reduction is primarily an effect of the increased stiffness.

However, ratios of recorded base shears in the structure with ADAS elements to base shears in the moment-resisting frame without ADAS elements were in the range of 0.6 to 1.25. Thus, the base shear in the ADAS frame was in some tests larger than the shear in the moment frame, as expected in the case of inelastic structures shown in Fig. 2(b). It should be noted again that the structure shear forces are primarily resisted by the ADAS elements and their supporting chevron braces (see Fig. 4). The ADAS elements yield in a pre-determined manner and relieve the moment frame from excessive ductility demands.

Various devices whose behavior is based on yielding properties of mild steel in forms of bell shaped steel devices, or honeycomb dampers, have been implemented in Japan. A simplified version of ADAS using multiple leaves of triangular plates (TADAS) were studied in Taiwan (Tsai, 1993).

3.2 Friction Devices

There are a variety of friction devices which have been proposed for structural energy dissipation. Usually friction systems generate rectangular hysteresis loops characteristic of Coulomb friction. Typically these devices have very good performance characteristics, and their behavior is not significantly affected by load amplitude, frequency, or the number of applied load cycles. The devices differ in their mechanical complexity and in the materials used for the sliding surfaces.

A frictional device located at the intersection of cross bracing has been proposed by Pall (1982, 1987) and has been used in six buildings in Canada. Figure 5 illustrates the design of this device. When seismic load is applied the interior deforms into a parallelogram and friction is produced at the bolt locations. Experimental studies by Filiatrault (1985) and Aiken (1988) confirmed that these friction devices could enhance the seismic performance of structures. The devices provided a substantial increase in energy dissipation capacity and reduced drifts in comparison to moment resisting frames without friction devices. Reduction in story shear forces were moderate as expected in inelastic structures. However, these forces are primarily resisted by the braces in a controlled manner and only indirectly resisted by the primary structural elements.

Sumitomo Metal Industries of Japan developed friction dampers for railway applications. Recently, the application of these dampers was extended to structural engineering. Several structures in Japan incorporate the Sumitomo friction dampers for aseismic protection. Figure 6 shows the construction of a typical Sumitomo friction damper. The device consists of copper pads impregnated with graphite in contact with the steel casing of the device. The damper can be installed in diagonal braces or parallel to the floor beams between the floor connections or in chevron brace arrangements (see Fig.6). Experimental studies by Aiken (1990) and Li (1995) resulted in conclusions similar to the study of friction bracing devices of Pall (1982). In general, the displacements are reduced in comparison to the unretrofitted moment resisting frames. However, the base shears and the accelerations are only slightly altered, sometimes increased, due to the strengthening of the new braces.

Constantinou and Reinhorn (1991a, 1991b) developed a friction device for application in bridge seismic isolation systems. Shown in Fig. 7 this device utilizes an interface of stainless steel in contact with bronze which is impregnated with graphite. The device bears a similarity with the Sumitomo device in terms of the materials which form the sliding interface.

A similar device fabricated by Tekton Company utilizes all the components used by Constantinou (1991a) in a different configuration (see Fig. 8). The device was used in concentric braces to retrofit a model r/c frame, as described later in the paper (Li and Reinhorn, 1995). The same damper was used in a masonry infill frame, between the frame columns and the masonry in a retrofit study (Rao, 1995).

Another friction device, proposed by Fitzgerald (1989), utilizes slotted bolted connections in concentrically braced connections. Component tests demonstrated stable frictional behavior. It may be noted that the sliding interface is that of steel on steel. Very recently Grigorian (1993) tested a slotted bolted connection (see Fig. 9) which was nearly identical to the one of Fitzgerald (1989) except for the sliding interface which consisted of brass in contact with steel. This interface exhibits more stable frictional characteristics than the steel on steel interface.

A more complex friction device (Energy Dissipation Restraint, EDR) combined with self centering capabilities provided by internal springs and end gaps (see Fig. 10) was developed by Flour Daniel Corp. (Nims, 1993). This device can develop X type hysteretic loops with restoring capabilities.

All the friction devices described above utilize sliding interfaces consisting of steel-on-steel, brass on steel, or graphite impregnated bronze on stainless steel. The composition of the interface is of extraordinary importance for insuring longevity of operation of these devices. Low carbon alloy steels (common steels) will corrode and the interface properties will change with time. Moreover, brass or bronze promote additional corrosion when it is in contact with low carbon steels. Only austenitic stainless steels with high chromium content do not suffer additional corrosion in contact with brass or steel (BSI, 1979).

4.3 Viscoelastic Devices

Viscoelastic dampers, made of bonded viscoelastic layers of acrylic polymers (see Fig. 11) have been developed by 3M Company Inc., and were used in wind and seismic applications. Examples are the World Trade Center in New York City (110 stories), Columbia See First Building in Seattle (73 stories), the Number Two Union Square Building in Seattle (60 stories), and General Service Administration Building in San Jose (13 stories).

The characteristics and suitability of viscoelastic dampers to enhance the performance of structures were studied by Lin et al. 1988, Aiken et al. 1990, Chang et al. 1991, and Lobo et al. 1993 using shaking table experiments. Figure 11 shows a typical damper and an installation detail in a concrete structure (Lobo, 1993). The behavior of viscoelastic dampers is controlled by the shear of the viscoelastic layers. The acrylic material exhibits solid viscoelastic behavior with storage and loss (shear) moduli dependent on frequency, temperature, and strain ratio. Variations of the stiffness modulus of 30% to 50% are expected for 10°C change at low frequencies and higher at larger frequencies.

In the aforementioned studies, 3M Company's dampers were used. Other devices developed by Lorant Group and used in the rehabilitation of connections were studied by Hsu, 1992. Hazama Corp developed devices using multi-layers of similar materials (Fujita 1991). Shimizu Corporation developed viscoelastic walls, in which solid thermoplastic rubber sheets are sandwiched between steel plates (Fujita 1991).

The use of dampers in elastic structures was shown to be efficient, in particular when the inherent damping of the structure is low (Aiken 1990, Chang 1991). The use of dampers in lightly reinforced structures was studied by Lobo (1993) and Foutch (1993). These studies indicate that the viscoelastic material dissipates large amounts of energy thereby reducing the demand for hysteretic energy dissipation in the columns and beams. The overall damping index (equivalent to critical damping in elastic structures) reaches 20% to 22%. However, the overall base shear in the structure has the tendency to increase or only minimally decrease, as expected in inelastic structures (see Fig. 2).

3.4 Viscous Walls

Viscous damping walls, consisting of a plate floating in a thin case made of steel plates (the wall) filled with highly viscous bituminous fluid (see Fig. 12), have been developed by Sumitomo Construction Company, Ltd., and the Building Research Institute in Japan. The walls were investigated by Sumitomo Construction Company (Arima, 1988) and are in use in a 78.6m high, 14-story building at the center of Shizuoka City, 150km west of Tokyo, Japan. Earthquake simulator tests of a five-story, reduced-scale building model and a four-story, full-scale steel frame building embedding such walls have been carried out (Arima, 1988). More recently, a three-story 1:3 scale reinforced concrete structure was retrofitted with viscous walls and was tested in a shaking table study (Reinhorn et al. 1994, 1995). The damping devices exhibited nonlinear viscous behavior with stiffening characteristics at high frequencies. In addition to their damping and strengthening characteristics, these devices can also be used as architectural cladding.

3.5 Fluid Viscous Dampers

Fluid dampers (see Fig. 13) consist of a stainless steel container filled with silicon fluid, a stainless piston with bronze orifices and accumulators (optional). The forces are generated by a pressure differential across the piston head. The resistance forces are proportional to the velocity of the piston or to the power of the velocity depending on the orifice configuration. Since the fluid is compressible, a reduction in the fluid volume is accompanied by the development of the restoring force (stiffness). This can be prevented or changed by use of an accumulator. Devices studied by Constantinou (1992, 1993) developed stiffness characteristics at larger frequencies only (>4 Hz) which were found to be beneficial in suppressing influences of higher modes. The fluid devices have stable properties over wide frequency and temperature ranges (0-25Hz, 0-50°C). The viscous characteristics varied 40% within 50°C range (~8% in 10°C).

The inclusion of fluid dampers in a steel structure in a shaking table study (Constantinou 1993) resulted in reductions of 30% to 70% of the story drift. In addition, the story shear forces were reduced by almost the same amount. The simultaneous reduction of displacements and forces indicates that the response of the structure without or with dampers was elastic. In another study (Reinhorn et al. 1995) the dampers were used to retrofit a damaged r/c structure in a shaking table study. The structure with dampers sustained inelastic deformations, reduced story drifts, and slightly increased story shears. However, the structure columns developed smaller shear forces. A more detailed presentation is furnished later in the paper.

4.0 CONSIDERATIONS IN THE DESIGN OF ENERGY DISSIPATION DEVICES

4.1 Combined Effects of Stiffening and Damping

It was mentioned above that the damping systems have both damping and stiffening characteristics. The predominant properties of analyzed systems are summarized below:

(a) *Metal yielding devices* have stiffening properties as well as the capability to dissipate energy (damping) as indicated by the name of some of these devices, ie. ADAS (added damping and stiffness) as shown in Fig. 16. Figure 17 shows the increase in structure capacity due to the stiffening - see also viscoelastic devices.

(b) *Friction devices* have strengthening properties as well as damping characteristics. The friction devices add "solid" strength to braces which are mounted until they slip when the forces reach the friction limit as shown in Fig. 14. As such, the structure is enhanced by as much strength as the slipping friction forces (see Fig. 17). At low deformations (before slip) the structure is stiffened, and at larger deformations just strengthened.

(c) *Viscoelastic devices* have stiffening properties (see Fig. 15) in addition to the damping characteristics (see Fig. 17). The stiffening properties may not be proportional to the damping and is dependent on the frequency. However, for a given constant frequency, the properties are proportional to the geometrical design of the polymer layers. VE dampers increase the capacity of a structure in the inelastic range as well as reduce demands due to damping enhancement (Lobo, 1993).

(d) *Viscous walls* behave more like VE dampers (see Fig. 17) showing stiffness at low frequencies, which further increases with the increase in velocity of motion. The high stiffness of these walls may produce, along with the desired effects of deformation reduction through seismic demand reduction, increase in story shears due to the capacity enhancement effects.

(e) *Viscous fluid devices* may have stiffening properties along with damping properties if so desired by design. These devices have the flexibility of providing the desired stiffness or cancelling the undesired one by initial design.

In summary, the damping devices, can produce stiffening characteristics that enhance structural capacity, sometimes with the undesired effects of shear force increases (see Fig. 17). Only the fluid viscous dampers can eliminate the stiffness contribution, if so desired. All types of damping devices reduce the seismic demand and the deformation response accordingly (see Fig. 17).

4.2 Implication of Use of Energy Absorbing Devices

The damping devices installed in braces or infill walls can produce substantial reductions to interstory drift and absorb energy that would otherwise damage the structural components, columns and beams, through hysteretic behavior. However, the retrofitted structure needs to transfer the forces through the braces. The placement of damping devices and the design of their connections require careful attention as noted:

(a) The stiffness component of the forces in the braces is in phase with the displacements, and therefore, maximum forces, in the structural system. This action can be viewed as the "conventional brace" action which increases lateral and axial forces in the columns where these braces are connected. A staggered arrangement of dampers over many stories of a moment resisting frame can increase axial load in columns and reduce their moment capacity making them more vulnerable to damage. A more rational arrangement of braces with continuity of force transfer to the foundations, may eliminate such an undesired effect.

(b) The damping component of the bracing system, related to the velocity is usually out of phase with the stiffening and lateral stresses in the structure (Constantinou, 1993, Reinhorn, 1995). Therefore, the maximum damping forces usually materialize when other stresses in the structural system are at zero. Therefore, the connections and other force transfer elements can be designed independently for damping forces and for the other lateral loads. Although some in phase effects can be noticed at very low frequencies, these are not dominant in the design (Reinhorn, 1995).

In conclusion, attention should be paid to the load transfer path resulting from the retrofitted system with dampers. The positive effects of drift reduction and energy dissipation may be inadvertently cancelled by the increased axial forces in columns which reduces their lateral moment capacity, in particular in taller structures.

4.3 Selection Procedure for Damping Devices for Seismic Retrofit

The selection procedure for damping devices is based on an appropriate yet single initial estimate of damping requirements using energy principles in an *elastic* approach. This is complemented by an inelastic push-over analysis combined with a composite spectrum approach (Reinhorn

1995). This method is the basis of a new methodology proposed by ATC-33.02, 1995 guidelines (in preparation at current time). The following outlines the suggested procedure:

(a) Based on a single mode approach, the critical damping ratio is related to the damping constant of each of the damping devices as follows:

$$\xi_d = \sum^{N} [C_{di} (f_{ik} - f_{i-1,k})^2 \cos^2 a] / \{2 w_k \sum^{N} [m_i f_{ik}]\}$$

where, ξ_d is the increase in the damping ratio, f_{ik} is the fundamental modal shape, a is the angle of inclination of the braces, w_k the fundamental frequency, and C_{di} is the viscous damping constant of all devices at one floor, N being total number of floors. Assuming a linear mode distribution and identical number of dampers at all floors, the desired damping constant for each device is obtained from the above equation:

$$C_{di} = 2 \pi \xi_d W_T(2N + 1) / 3 g T_k \cos^2 a$$

where, W_T is the total weight of structure, g the gravitational acceleration and T_k is its fundamental period. Assuming harmonic motion, the maximum story damping force is obtained:

$$F_{di} = 2 \pi C_d d_{max} \cos a / T_k$$

where, d_{max} is the maximum interstory deflection. The characteristics determined here are only rough estimates based on initial structural properties and may be adjusted based on the expected structural properties. For a complete design, however, further steps need to be taken.

(b) Prepare an analytical model of the building and include the stiffening characteristics of the assumed dampers. An inelastic push-over analysis (Reinhorn, 1995) should be made to determine the capacity of the structures. It is desirable to perform the analysis without the dampers and with the dampers separately to obtain the influence of the retrofit. Functions of structural capacity for each story and for the entire structure (base shear versus top story drift) can be prepared similarly to those shown in Fig.1(a) or (b).

(c) The seismic demand can be determined from design acceleration and displacement spectra to produce the composite function as shown in Fig.1(c) and (d). The composite spectrum should be adjusted to reflect the assumed damping contribution by suitable reduction of spectral ordinates by suggested damping reduction factors (NEHRP, 1994).

(d) The expected structure response can be obtained at the intersection of the capacity diagram developed in (b) with the seismic demand from (c). The results may show differences from the assumed values, i.e., initial and service period, or effective damping ratio. Following suitable iteration the resulting response characteristics will indicate the deformations and forces in the retrofitted system.

(e) Following the preliminary response evaluation, an evaluation of force transfer should be made by either push-over (monitoring influence of stiffening and strengthening on the elements of structure) or by a time history analysis to determine also the contribution of damping force transfer.

5.0 CASE STUDIES OF RETROFIT OR LRC FRAMES WITH SUPPLEMENTAL DAMPERS

A comprehensive study of retrofit of a 1:3 scale lightly reinforced concrete frame was carried out at NCEER involving shake table tests and analytical studies (Reinhorn et al. 1995a, b). The structure was damaged by prior testing, then was conventionally retrofitted by concrete jacketing (Bracci et al. 1992b) and damaged again with severe shaking table excitations. Finally, the structure was retrofitted using various damping systems and studied experimentally and analytically (Reinhorn, 1995a). The structure, made of two parallel, three story, three bay, frames, was retrofitted by bracing the mid bay with the following devices:

(a) viscoelastic (3M Corporation)
(b) fluid (Taylor Devices)
(c) friction (Teckton Company)
(d) friction (Sumitomo Industries, Inc.)
(e) viscous walls (Sumitomo Construction Company)

The results of the study are the subject of several NCEER reports (Lobo, 1993, Reinhorn et al. 1995a,b, Li et al. 1995). The retrofitted structure was subjected to numerous moderate and extremely strong earthquakes (i.e., El Centro, 1940 x150%; Taft 1952 x200%; Hachinohe 1968 x150%, etc.). The structure's deformations were maintained below damage limits while forces reached the yielding levels during all earthquakes. The story drift was reduced compared to the unretrofitted structure by 50% to 70%, depending on the device and its stiffening characteristics (see Fig. 19a). The viscous walls produced substantially larger stiffness increase than all other devices, based on the current design. It also produced the largest reduction in deformations, although all the other devices produced reductions of the same order of magnitude. These reductions fit the expectations based on the capacity-demand criteria illustrated in Figs. 2 and 17. However, the overturning moments and the story shears obtained ranged from 0.8 to 1.35 times the unretrofitted moment frame (see Fig. 19b, and c). This is due to the expected effect of stiffening and strengthening of the structure. It should be noted that the largest part of the increased load is transferred through the damping braces, while the shear forces in the columns are reduced (see Fig. 19c).

The studies showed that the conversion of the moment frame into a braced frame does not increase substantially the axial forces in the columns (Reinhorn, 1995a), however, in a taller structure this change might be different and detrimental. The retrofit reduces the deformation response with a minor increase in the base shear, that may only affect the existing foundations, when the retrofit technique offers stiffening and strengthening. However, even in such cases, the benefits in reducing deformations may justify allowing minor base shear increases. A careful analysis of the final solution is recommended in all cases of retrofit, to insure that no element is adversely affected.

A recent study using masonry infills and friction dampers for seismic retrofit indicated similar influences as obtained in the above case study (Rao, 1995). Shaking table studies at University of Illinois of retrofit using VE dampers are completed and final reports are expected (Foutch, 1993).

6.0 FULL SCALE IMPLEMENTATIONS OF RETROFIT USING SUPPLEMENTAL DAMPERS

Damping devices are currently used in seismic retrofit and for other applications in building structures throughout the world. Table 2 summarizes the applications in North America, with emphasis on retrofit which constitute approximately 45% of the total uses. The majority of retrofitted cases include lightly reinforced concrete frame structures. Yielding steel devices were used in the majority of applications in Mexico and U.S., probably due to the known reliability of steel behavior in civil engineering applications. The most documented retrofit of an RC frame in U.S. (Fierro, 1993) is the Wells Fargo Bank Building in San Francisco, retrofitted after the Loma Prieta earthquake using seven ADAS dampers mounted with Chevron braces (see Fig. 21).

A recent project currently under construction will use viscoelastic dampers in a reinforced concrete structure for a Navy office supply building in San Diego. The retrofit controls the building *deformations* to reduce the potential damage. The installation work will cause minimal disruption to the use of the building. The connections of dampers to the reinforce concrete elements will be done through metal braces and specially jacketed column connections (UB/News, 1995). A desired damping increase of 15% was obtained with a small number of dampers.

Additional projects are under design and the new seismic guidelines (SEAONC, 1993, NEHRP, 1994, and ATC33.02) provide new directions for such uses. The newer guidelines, however, promote the reduction of seismic demands through increased damping and pays less attention to the stiffening and strengthening characteristics of the damping devices.

Fluid viscous dampers were used in a retrofit of a hotel building (see Table 2) and are currently considered for use in several other retrofit projects of RC structures. Since these devices have little or no effect on the stiffening of structures, the current codes guidelines are well suited for their use.

7.0 DISCUSSIONS AND RECOMMENDATIONS

The seismic retrofit of lightly reinforced concrete structures can benefit immensely from use of supplementary damping devices. These devices can control and reduce the inelastic deformations and remove the demand for energy dissipation from the gravity load supporting members. The retrofit can be done by-and-large with minimal interference in the existing structural system and minimal disruption to the structure.

However, important issues should be addressed by the designer and further research is still necessary to clear complex issues:

(a) The durability and longevity of friction devices is an issue for the design professionals and researchers. Metal to metal slip friction characteristics may be altered due to corrosion if mismatched materials are used at the interface. Steel on steel, bronze or brass on steel may produce increased corrosion. The development of new materials, as used in automotive industries, is suggested and should be encouraged (research needed). Design guidelines and limitations would be explored for the current materials (joint efforts of researchers and engineering professions needed).

(b) Most damping systems are suitable when load reversals are expected. For shock loadings only specially designed dampers can be used. Structures need to be verified for expected seismic action specific to the construction site. Near fault construction needs to have shock protection as well as damping energy dissipation (research needed).

(c) Viscoelastic materials experience property changes at small variations of temperatures. The design process needs to consider such variations to ensure safety. (guidelines are required). Other means of temperature compensation or new materials less sensitive can be developed (research needed).

(d) The retrofit design of any system, using dampers, as well as using conventional methods needs to be verified for its total integrity and not just for individual components. The interaction of new and old components may render one to be inefficient and the other to be unsafe. Proper integral design can prove to be beneficial to the structure. The capacity - demand approach using the composite spectra seem to provide a good frame for such integral evaluations. Case studies need to be explored further to clear the contribution of in-phase damping forces, the influence of extremely large damping on inelastic response of LRC elements and simultaneous contribution of hysteretic and viscous damping (research and practicing engineering community must join in development and research).

(e) The current damping systems use entirely passive components. If the response can be measured in real-time and fed back into the damping device, an adjustment can be made to obtain an adaptable or semi-active damper. These types of devices might be applicable for both shock and damping energy dissipation. (Researchers and industries need to join forces).

(f) Passive mass dampers which produce also energy dissipation by transfer of power between modal components, can be coupled with sensing and real-time processing to produce hybrid dampers which may adjust properties to protect LRC structures against multiple hazards such as wind and earthquake simultaneously (Researchers and industries need to join forces).

Finally, new techniques and materials can provide future developments which may allow less expensive, faster, more durable and more versatile retrofit.

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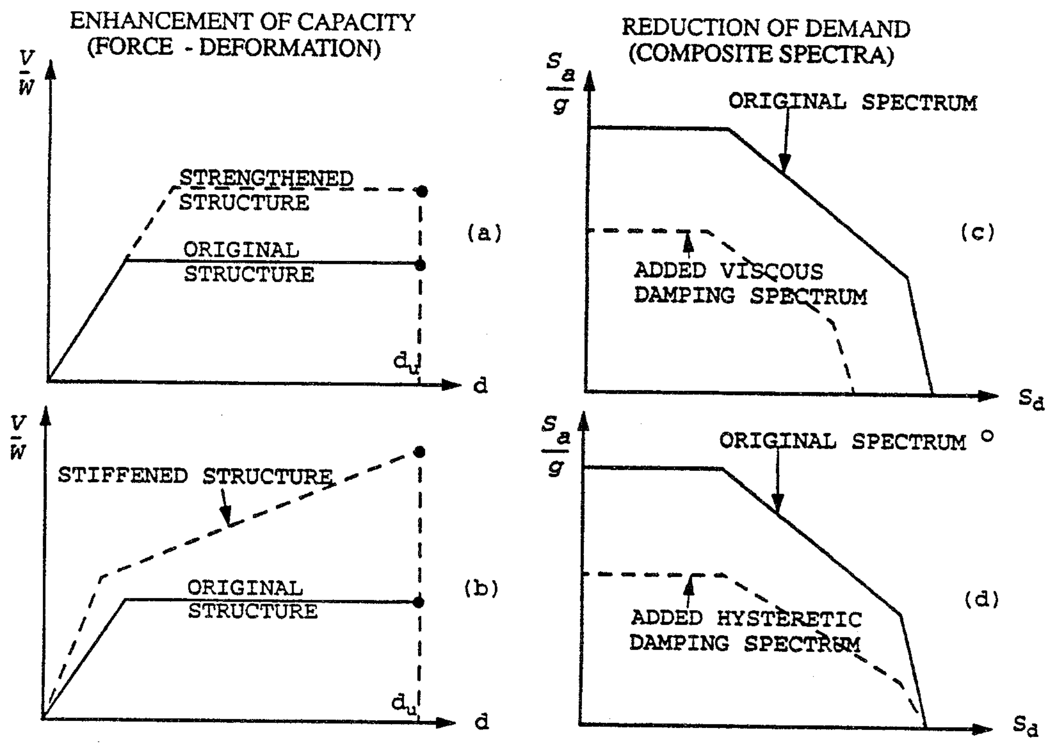
Table 1 Passive Energy Dissipation Systems in U.S.

Classification	Principle of Operation	Materials/ Technologies	Characteristics	Development in U.S.	Applications in U.S.	U.S. Industry
Hysteretic	Friction	Metal to metal or non-metal contact	Energy dissipation/ strength enhancement	Devices developed, recently marketed	None	Small, recently established companies
	Yielding Metal	Steel, lead	Energy dissipation/ strength enhancement	Devices developed, recently marketed	Seismic hazard mitigation	Small, recently established companies
	Phase transformation of metals	Shape-memory alloys	Energy dissipation/ strength enhancement	Some research conducted	None	Some small companies in medical fields
Viscoelastic	Deformation of viscoelastic solids	Viscoelastic polymers	Energy dissipation/ stiffness enhancement	Developed and marketed for over 25 years	Wind vibration control, seismic hazard mitigation	3M
	Deformation of viscoelastic fluid	Highly viscous fluids	Energy dissipation/ stiffness enhancement	None	Vibration and seismic isolation systems	None
	Fluid orificing	Fluids/ advanced orifice designs/ fluid sealing	Energy dissipation	Available since 1925, significant developments for military applications	Seismic hazard mitigation/ elements of seismic isolation systems	Taylor Devices Enidine
Compressible Fluid	Fluid orificing and pressurization	Fluids/ advanced orifice design/ high pressure sealing	Preload, constant restoring force, energy dissipation	Available since 1955, significant developments for military applications	Military and industrial/ designs developed for seismic hazard mitigation	Taylor Devices
Tuned Systems	Tuned mass-spring-damper oscillator	Mass-spring-fluid damper	Enhancement of damping	Developed and originally applied in U.S.	Wind vibration control	MTS Systems
	Tuned liquid oscillator	Water tanks	Enhancement of damping	Some research conducted	None	None

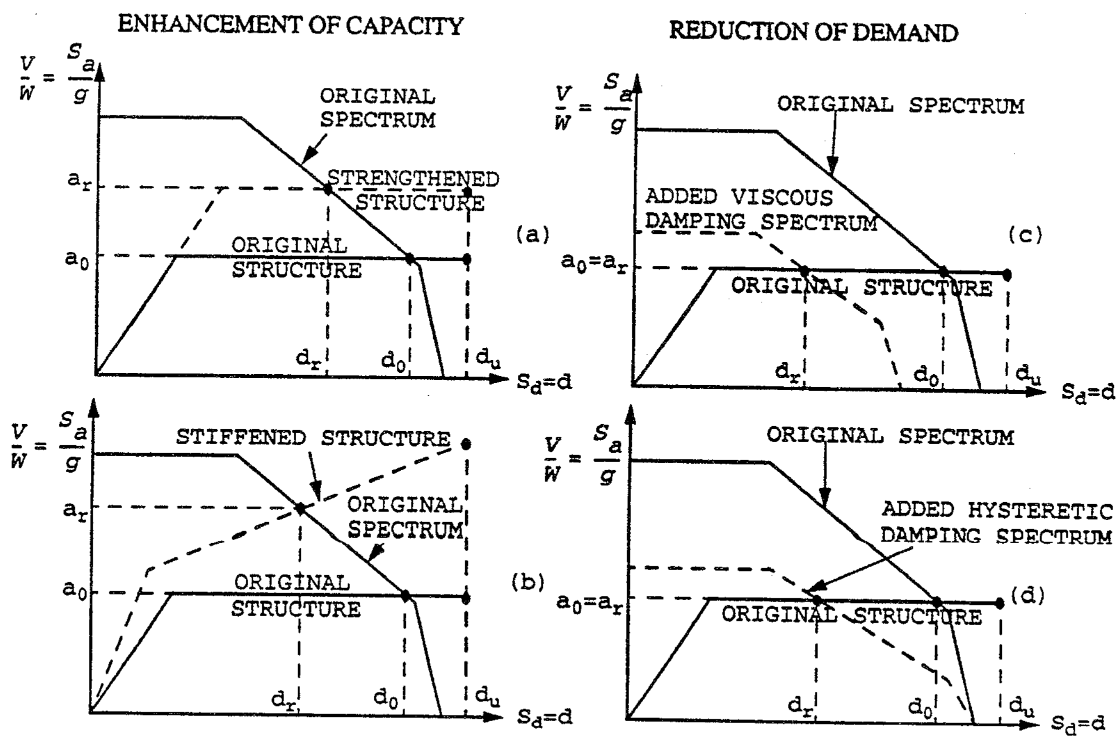
Table 2 Applications of Damping Devices in North America

Building						Passive Device			
Name(Construction Year)	Location	Size	Type	Status	Year	Type	#	Size(for./stroke)	Fabricator
World Trade Center (c.1969)	NewYorkCity/USA	110 st. x275000sq.m	steel	new	1969	Viscoelastic	20k	100kN	3M Corp.
CN / Tower (c.1973)	Toronto/Canada	553m tower	r/c tower	new	1973	TMD	1	n/a	n/a
Citicorp Center (c.1978)	NewYorkCity/USA	69 st. x180000sq.m	steel	new	1978	TMD	1	4000kN/900mm	MTS Corp
Columbia SeaFirst Bldg.(c.1982)	Seattle/USA	73 st.	steel	new	1982	Viscoelastic	260	n/a	3M Corp.
Two Union Square Bldg (c.1988)	Seattle/USA	60 st.	steel	new	1988	Viscoelastic	16	n/a	3M Corp.
Concordia Library (c.1990)	Montreal/Canada	10+6 st. x50000 sq.m	r/c frame	new	1990	Pall(friction)	143	700kN/	PallDynamics
School Bldg.	Phoenix/USA	2 st.	n/a	new	1992	Viscoelastic	n/a	n/a	Lorant Group
Canadian Space Agency (c.1993)	Montreal/Canada	3 st. x31590sq.m	steel	new	1993	Pall(friction)	58	500kN	PallDynamics
Casino de Montreal(c.1993)	Montreal/Canada	8 st.	steel	new	1993	Pall(friction)	32	1800kN	PallDynamics
San Bernardino Hospital (c.1995)	SanBernardino/USA	n/a	steel	new	1995	Fluid/Viscous	233	1400kN/1200mm	TaylorDevices
Pacific Bell/911 N.Cal (c.1995)	Sacramento/USA	4 st. x14300sq.m	steel	new	1996	Fluid/Viscous	62	140kN/100mm	TaylorDevices
Gorgas Hospital	PanamaCity/Panama	n/a	r/c frame	retrofit	1975	Friction	2	n/a	n/a
JohnHancockTower (c.1975)	Boston/USA	58 st. x210000sq.m	steel	retrofit	1977	TMD	2	3000kN/900mm	MTS Corp
Ecole Polyvalante (c.1967)	Sorel/Canada	3 st. x40000sq.m	precast	retrofit	1990	Pall(friction)	64	355kN	PallDynamics
Izazaga #38-40(c.1970)	MexicoCity/Mexico	13 st. x14500sq.m	r/c frame	retrofit	1990	ADAS(steel)	200	n/a	custom
Cardiology Hospital Bldg.(c.1970)	MexicoCity/Mexico	n/a	r/c frame	retrofit	1990	ADAS(steel)	90	n/a	custom
Wells Fargo Bank (c.1967)	San Franscisco/USA	2 st. x1300sq.m	r/c frame	retrofit	1992	ADAS(steel)	7	680kN/20mm	custom
Reforma #476 / IMSS (c.1940)	MexicoCity/Mexico	10 st.+basem	r/c frame	retrofit	1992	ADAS(steel)	400	n/a	custom
Santa Clara County Bldg (c.1976)	San Jose/USA	13 st. x32500sq.m	steel	retrofit	1993	Viscoelastic	96	500kN	3M Corp.
Woodland Hotel	Woodland/USA	5 st.	masonry	retrofit	1995	Fluid/Viscous	16	230kN/100mm	TaylorDevices
Warehouse / US Navy (c.1960)	San Diego/USA	3 st.	r/c frame	retrofit	1996	Viscoelastic	36	900kN	3M Corp.



Various rehabilitation procedures
(a)Strengthening, (b)Stiffening, (c)Added viscous damping, (d)Added hysteretic damping

Fig. 1 - Effect of rehabilitation procedures.



Result of rehabilitation - single parameter influence
(a)Strengthening, (b)Stiffening, (c)Added viscous damping, (d)Added hysteretic damping

Fig. 2 - Effect of rehabilitation - single parameter influence.

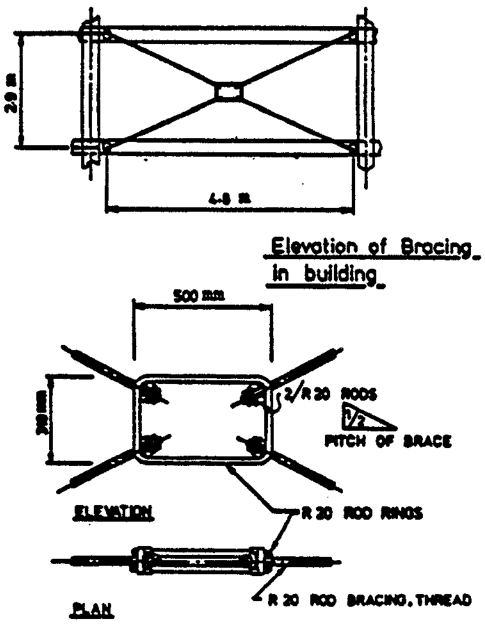


Fig. 3 - Details of yielding steel bracing system (from Tyler, 1985).

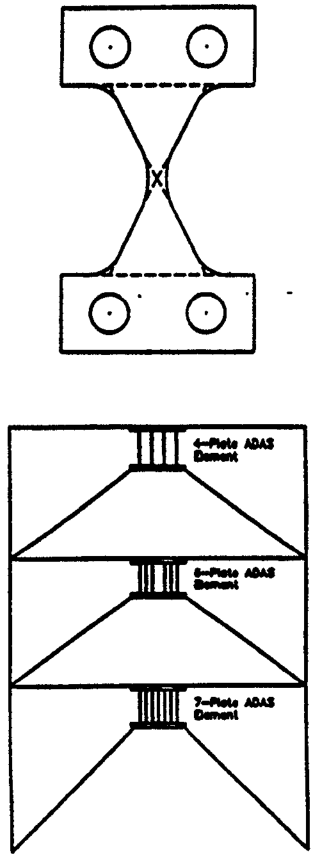


Fig. 4 - ADAS X shaped steel plate and installation details (from Whittaker, 1990).

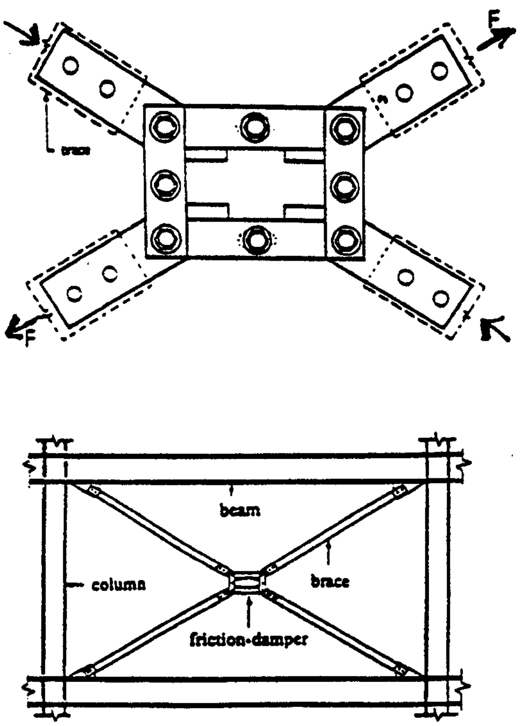


Fig. 5 - Friction damper of Pall Dynamics (from Vesina, 1992).

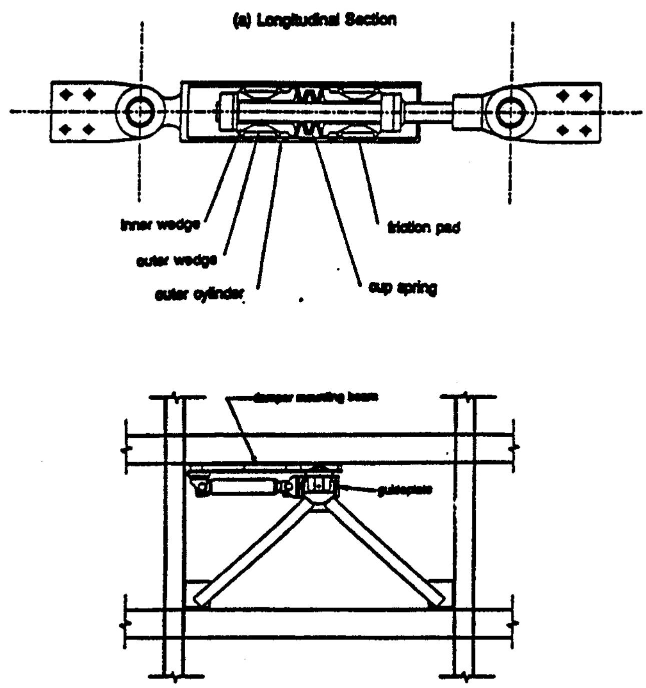


Fig. 6- Sumitomo friction damper and installation details (from Aiken, 1990).

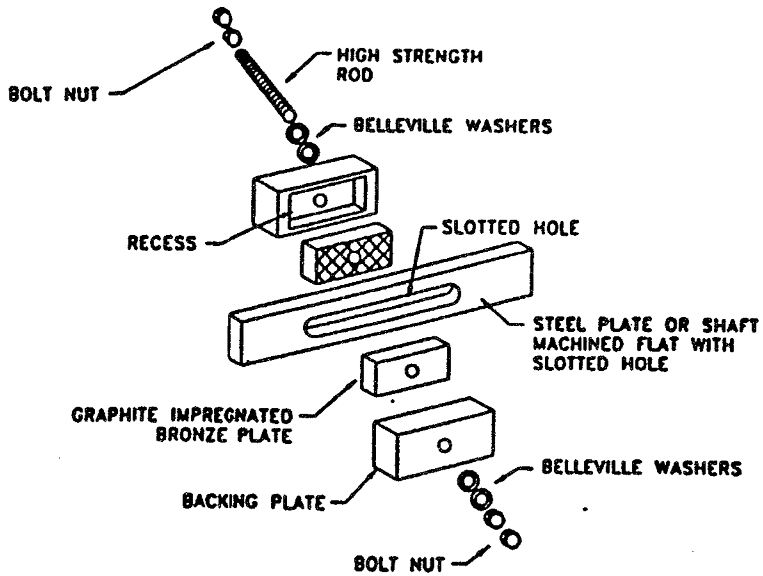


Fig. 7 - Friction assembly in displacement control device (from Constantinou and Reinhorn, 1991).

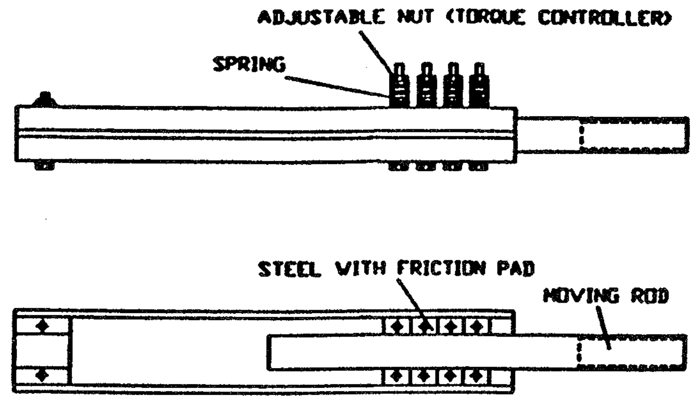


Fig. 8 - Tekton friction control device (from Li, 1995).

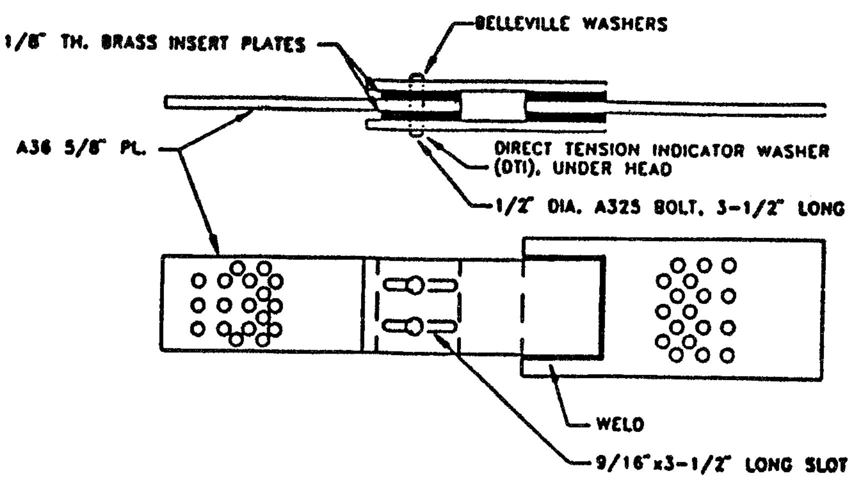


Fig. 9 - Slotted bolted connection (from Gregorian 1993).

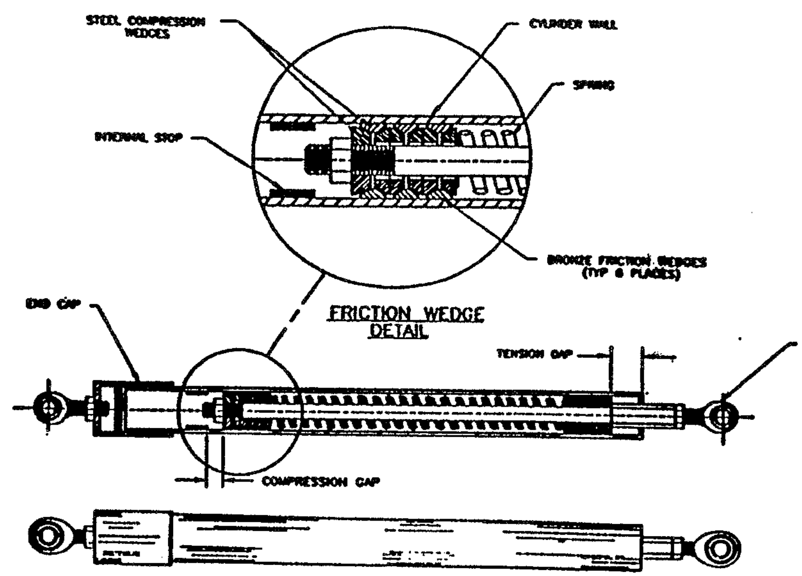


Fig. 10 - Energy dissipation restraint (from Nims, 1993).

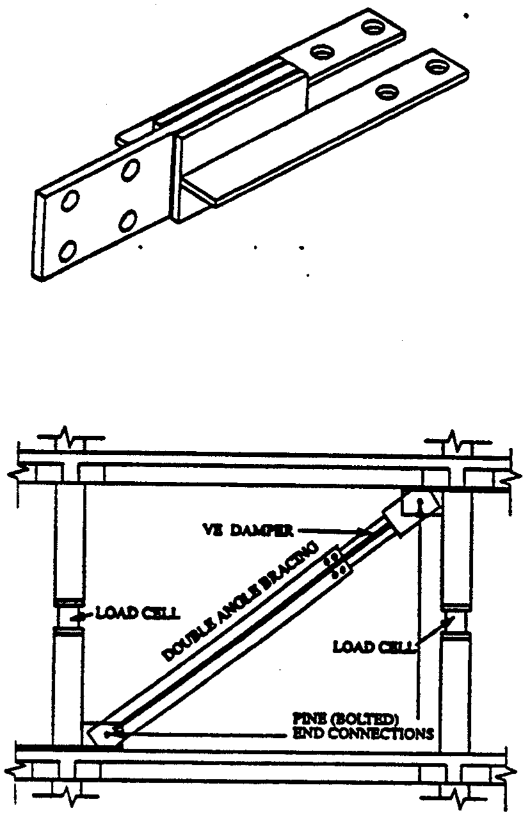


Fig. 11 - Viscoelastic damper and installation (Lobo, 1993).

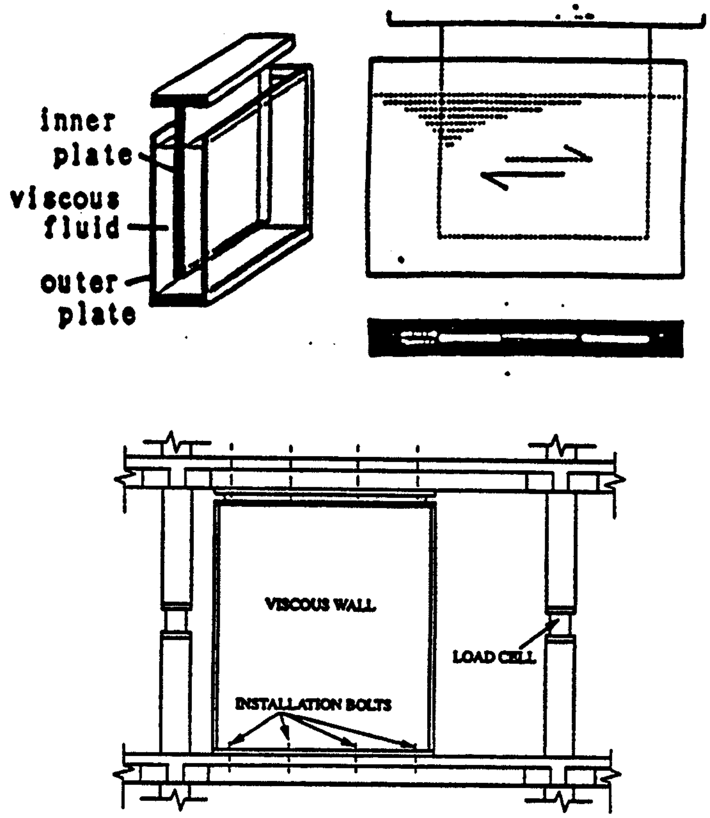
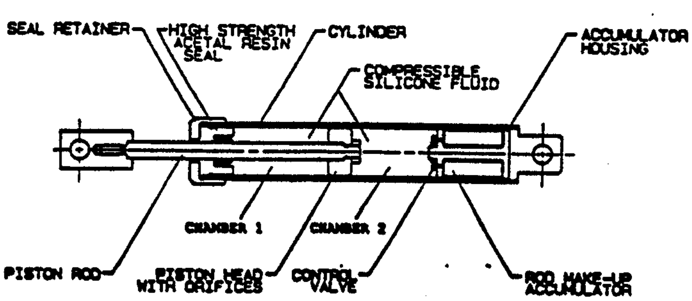
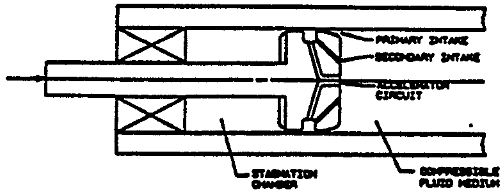


Fig. 12 - Viscous walls and installation (Reinhorn, 1995b).





Fluidic Control Orifice

Fig. 13 - Fluid viscous damper (Constantinou, 1992).

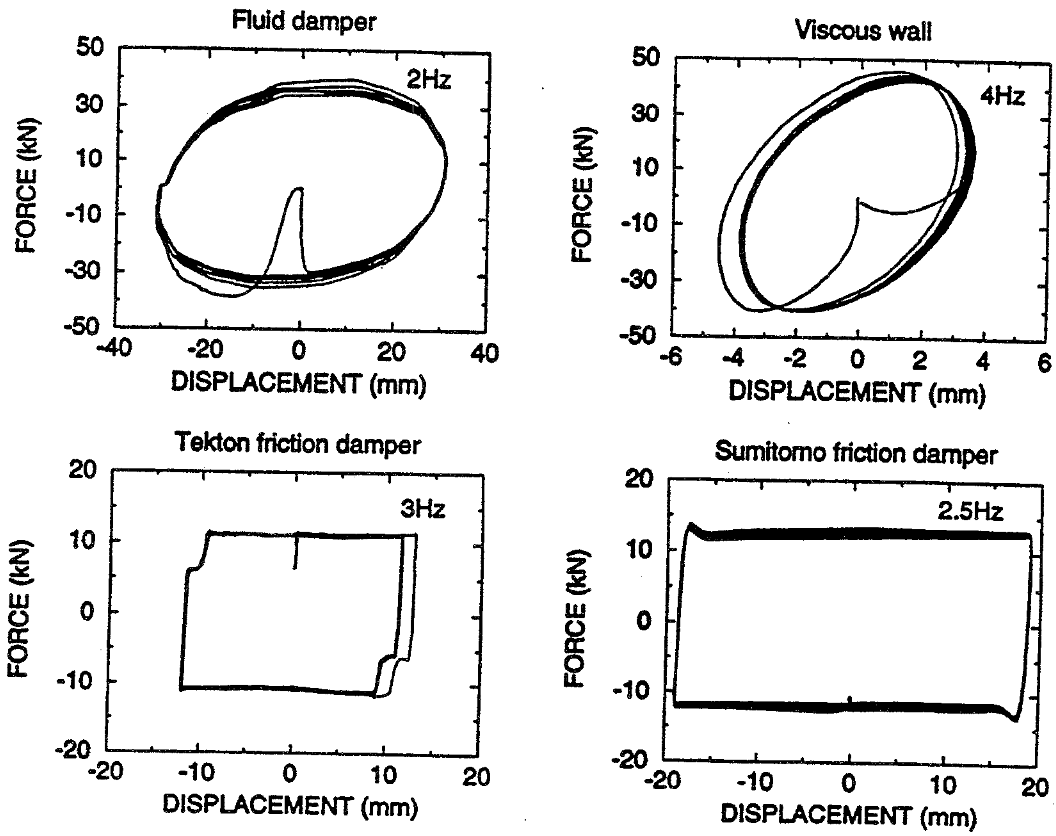


Fig. 14 - Force displacement characteristics of damping devices (Reinhorn, 1995c).

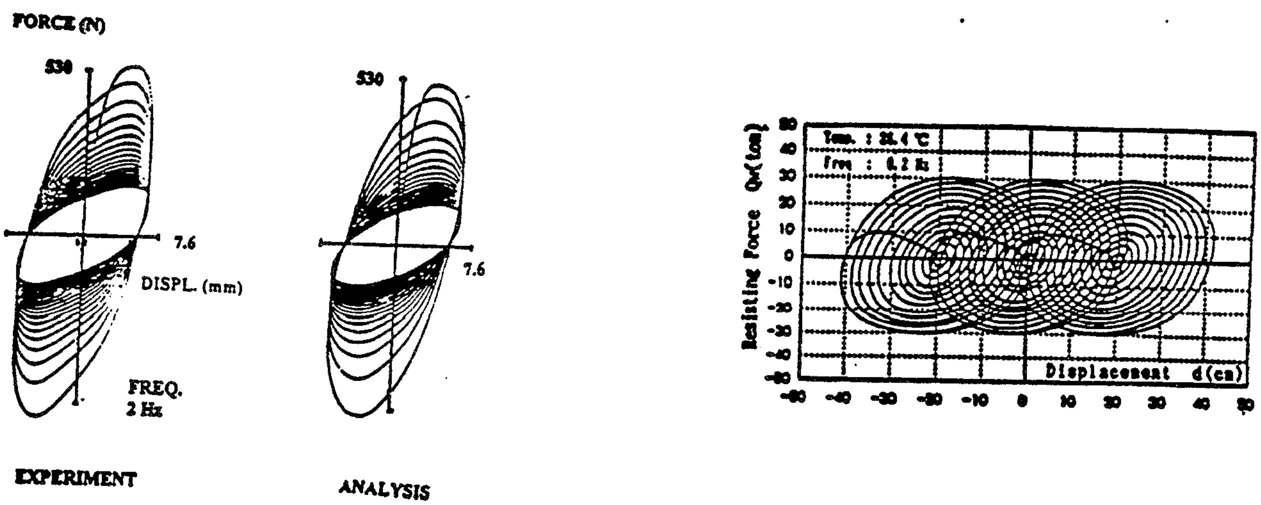


Fig. 15 - Force displacement loops of (a) VE dampers (Kasai, 1993) and
(b) viscous walls (Miyazaki, 1992).

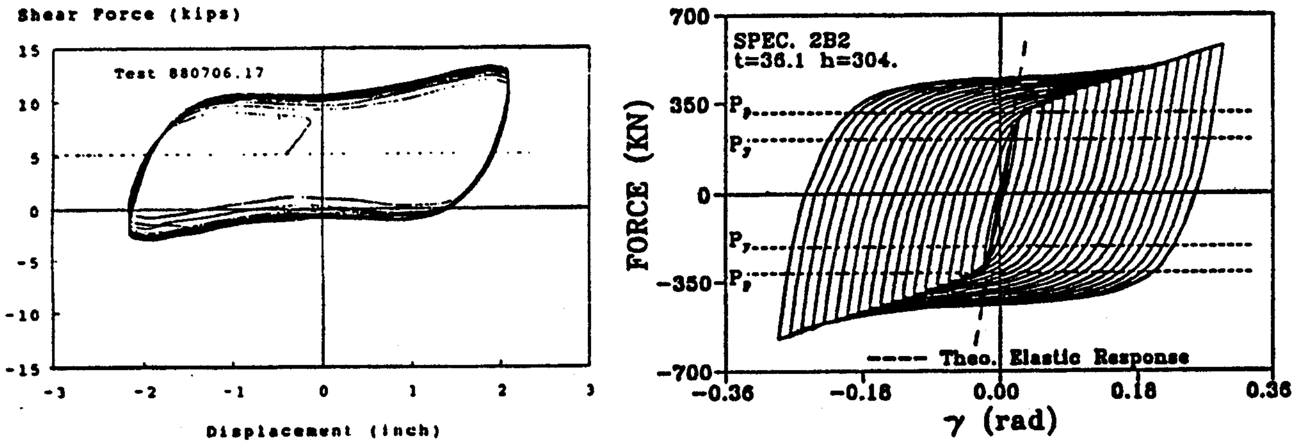


Fig. 16 - Hysteretic loops of metal yielding devices (a) ADAS (Whittaker, 1991) and (b) TADAS (Tsai, 1992).

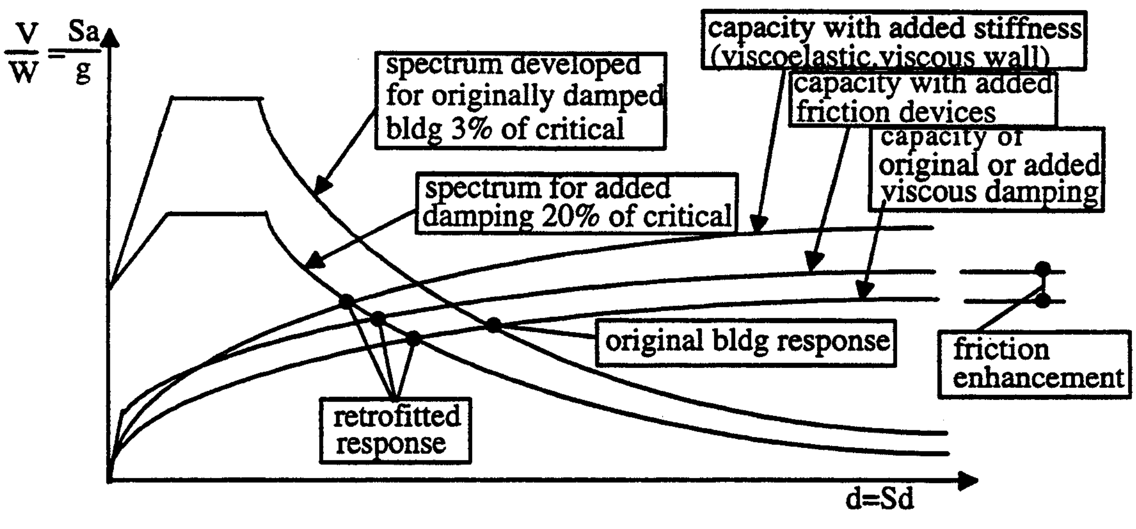


Fig. 17 - Capacity demand diagram with dampers (Reinhorn, 1995c).

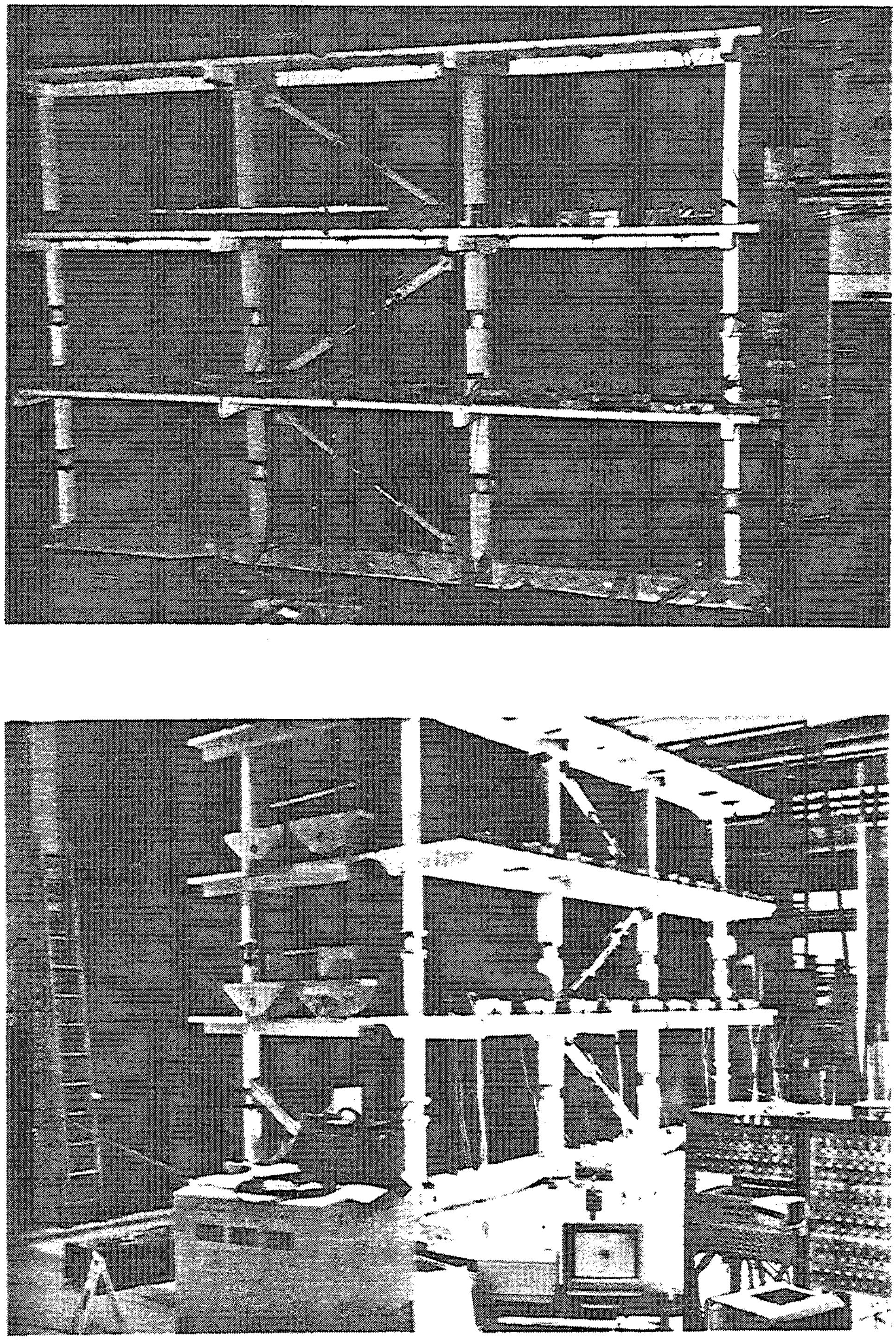


Fig. 18 - Retrofitted structure with damping braces.

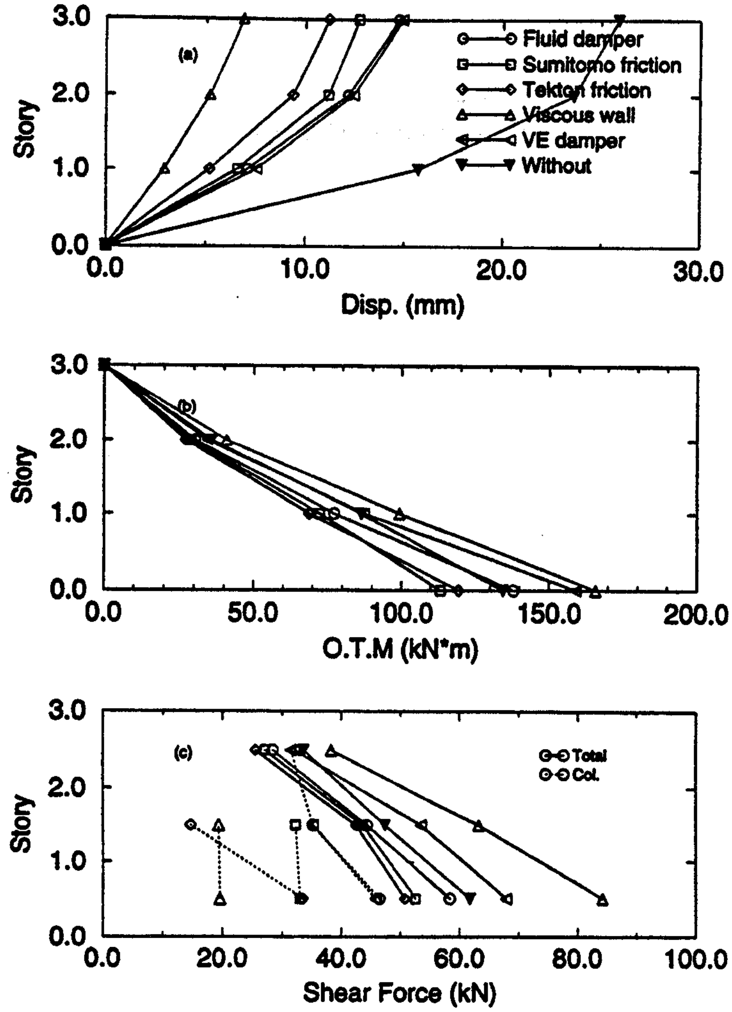


Fig. 19 - Experimental response of retrofitted structure with supplemental dampers.

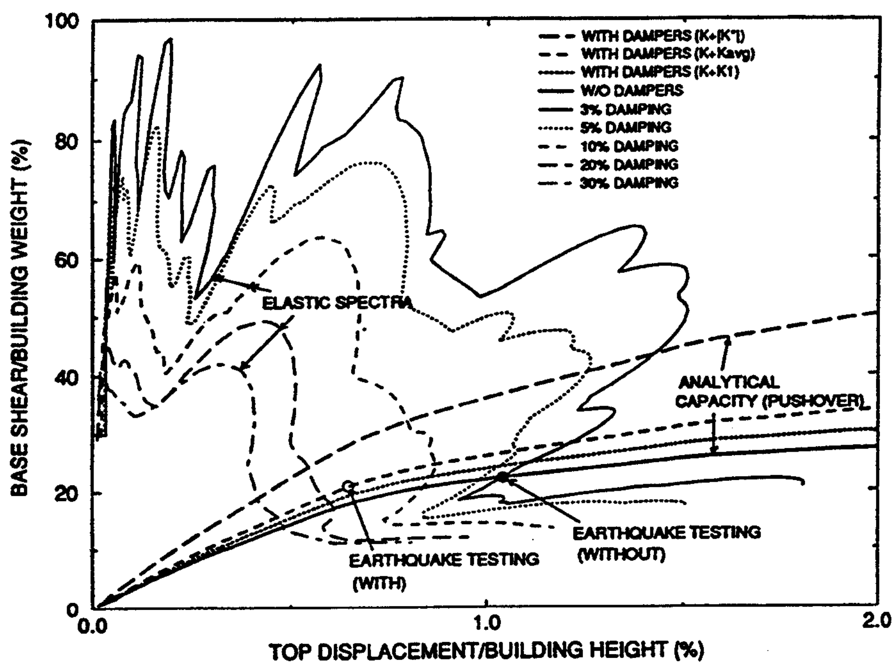
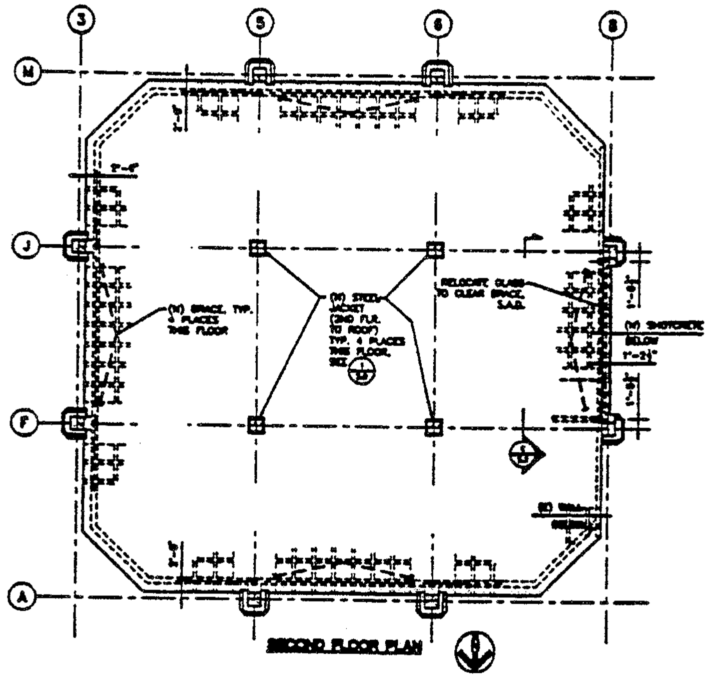
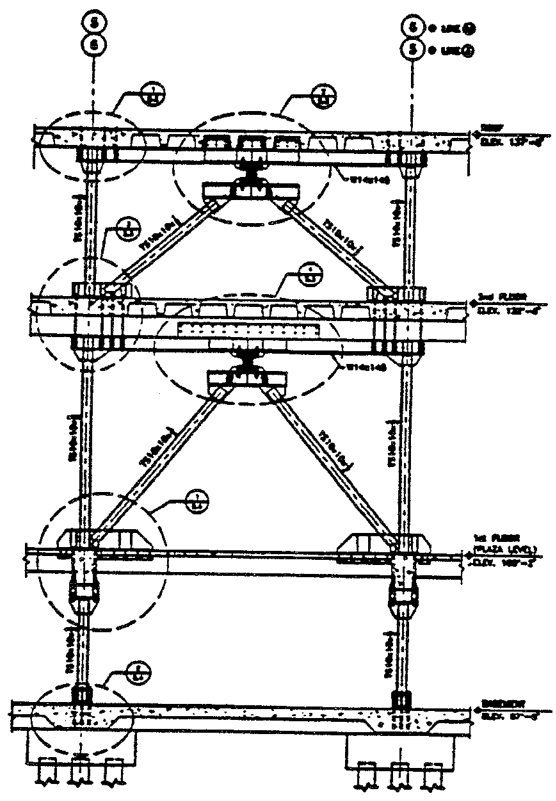


Fig. 20 - Capacity demand in retrofitted structure (experimental) (from Reinhorn, 1995a).



. SECOND FLOOR PLAN W/BRACE LOCATIONS



NEW BRACED FRAME W/ADAS ELEMENTS @ LINE A

Fig. 21 - Retrofit of Wells Fargo Bank (from Fierro, 1993).

USE OF FIBRE REINFORCED COMPOSITES FOR STRENGTHENING AND REHABILITATION OF CONCRETE STRUCTURES

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ABSTRACT

Fibre reinforced composite materials are rapidly being introduced into a variety of civil engineering applications. These materials have been found to be particularly attractive for applications involving the strengthening and rehabilitation of existing reinforced concrete structures. In this paper, the various methods of repair and rehabilitation of concrete structures using fibre reinforced composites are reviewed. These repair techniques include column retrofitting with composite wraps, beam and girder rehabilitation with bonded composite laminates, and repair using composites for external prestressing. Various field applications are presented, and research issues related to the rehabilitation of lightly reinforced concrete frames are discussed.

1.0 INTRODUCTION

The introduction of fibre reinforced composite materials in civil engineering structures has progressed at a very rapid rate in recent years. These high-performance materials, which consist of high strength fibres embedded in a plastic matrix, have unique properties which make them extremely attractive for a wide range of structural applications. Fibre reinforced composites are non- corrosive; they have high strength-to-weight ratios, possess good fatigue behaviour and low relaxation, are electromagnetically neutral, and also allow easy handling and installation. Moreover, as the fibre types (glass, carbon or aramid) and fibre volumes can be combined in innumerous ways with a variety of available polymeric matrices, their overall mechanical properties can be tailored to provide optimum solutions to a variety of structural problems.

The past and potential future uses of fibre reinforced composites in structural engineering have been documented in many keynote lectures [1], review articles [2-4], monographs [5-7] and conference proceedings [8-10]. These references provide an excellent background on the properties of composites, as well as the state-of-the-art regarding research and development in this emerging new field. The rapidly expanding body of literature in this area, along with the corresponding increase in level of activity, confirm the fact that these new materials are progressively gaining wider acceptance by the civil engineering community.

One area where the use of fibre reinforced composites has attracted considerable interest is in the strengthening and rehabilitation of reinforced concrete structures. Many bridges in North America are structurally deficient due to deterioration and corrosion, while others are functionally obsolete because of service loads and traffic volumes that greatly exceed their initial design loads. The situation regarding deterioration is quite similar for many parking structures, as well as for our immense aging municipality infrastructure. Although fibre reinforced composites are generally more expensive than conventional construction materials, retrofitting using composite patching and wrapping instead of bonded steel plates or steel jackets can nevertheless be economically viable due to the offsetting savings in labour costs [2]. Indeed, fibre reinforced composites are expected to become the materials of choice in the future, for example, for repairing earthquake damaged bridges and buildings [11].

This paper focuses on the strengthening and rehabilitation of reinforced concrete structures using fibre reinforced composites. Various methods of strengthening and repair for beam and column structures are reviewed and various applications, including seismic retrofitting, are discussed. Finally, questions associated with the rehabilitation of lightly reinforced concrete frames are addressed. An extensive, yet not exhaustive, bibliography is provided and the interested reader is referred to this for more detailed information on the current and potential uses of composites in civil engineering applications.

2.0 COLUMN REHABILITATION WITH FIBRE COMPOSITE WRAPS

The use of fibre composite wraps to strengthen and repair existing reinforced concrete columns is undoubtedly the composites application which has generated the most interest to date among structural engineers [12-19]. With this technique, external reinforcement and confinement is provided by wrapping unidirectional composite sheets or straps around the concrete columns. This method is of practical interest because the lay-up of the sheets is rather easy; it does not require specialized tools, and the epoxies employed cure at room temperature. Furthermore, since the composite wraps are thin and flexible they can easily conform to any column shape or geometry.

Both active and passive confinement are possible with this method. Active confinement is achieved by pressure grouting with either epoxy or cement in the gap between the inner layer of sheet and the column. This process induces reasonably large hoop strains in the composite wrap, which in turn ensures that the grouted pressure is maintained with minimal loss after the grout has hardened. The confinement thus provided greatly enhances the column behaviour, leading to significant increases in strength and ductility.

Typical results showing the effectiveness of passive composite wrapping are presented in Fig. 1. These are the results of an investigation where both glass fibre and carbon fibre wraps have been applied to round and square short plain concrete columns loaded in compression [16]. We observe here that, in all cases, composite wrapping significantly increases ductility, up to seven times that of the unwrapped specimens. In addition, for glass fibre wrapping of round columns with three layers, or one layer of carbon fibre sheet, the column strength is substantially increased (up to 50%). Strength increases for round columns as high as 70% were obtained with three layers of carbon sheet. The results for wrapped square columns show increases in ductility comparable to those obtained for the round columns. The maximum strength levels for these shapes, however, show very little improvement over those of the unwrapped specimens. This is to be expected since the confinement is less effective for rectangular shapes. However, the behaviour of rectangular shapes can be improved somewhat by rounding the corners of the columns before applying the composite wrap.

The effectiveness of using fibre composite wrapping to repair severely damaged columns is illustrated in Fig. 2. This is one of an extensive series of tests in our laboratory on conventional reinforced concrete columns (1200 mm in height and 300 mm in diameter), having various types of axial and shear steel reinforcement [17]. For the case presented in Fig. 2 the column was initially loaded beyond its carrying capacity and then unloaded, as indicated by the dotted curve. This resulted in extensive delamination of large pieces of concrete, as well as a slight buckling of the axial steel reinforcement. The damaged concrete was patched, and the column wrapped with three layers of carbon fibre sheet. The repaired column was then reloaded in compression. The behaviour obtained is shown as the solid curve in Fig. 2, which clearly demonstrates that the composite wrapping repair method is very effective for both improving ductility and restoring the structural integrity of damaged concrete columns.

The concept of retrofitting bridge columns to improve flexural and shear performance by appropriately using fibre composite jackets in critical regions is described in [12]. There, test results are reported for both large-scale flexural columns having longitudinal steel reinforcement lapped in the flexural plastic hinge area, as well as for shear columns loaded under double bending. Active confinement is provided in the flexural plastic hinge regions using pressure grouting as described above. Also, layers of composite wrap are also placed over the end portions of the plastic hinges to provide additional passive confinement in those regions where either high compressive strains are expected, or where the presence of lap- spliced longitudinal bars indicates the need for additional restraint. The regions between potential plastic hinges in columns subjected to high shear forces may also be strengthened with this same active/passive combination.

The experimental results in [12] show that properly designed composite wraps for reinforced concrete columns can inhibit lap- splice failures in hinge regions, enhance flexural ductility, and also provide sufficient shear strength to the extent that brittle shear failure modes are converted to inelastic flexural deformation modes. For rectangular columns, ductility and strength enhancement may be limited, however, unless curvature in the external composite wrap is provided to ensure sufficient confinement [4]. In [12] simple design models are also presented. These are shown to give conservative predictions of the stress levels in the composite wraps, and safe estimates of the flexural ductility and shear strength enhancement.

3.0 BEAM REHABILITATION WITH COMPOSITE LAMINATES

Important pioneering work on increasing the flexural capacity of existing reinforced concrete beams and girders by bonding composite laminates was initiated about ten years ago by Meier's group at the Swiss Federal Laboratories for Materials Testing and Research (EMPA) [2,20-22]. Extensive previous work at EMPA with bonded steel plates had indicated that, although this method proved to be successful, it has certain disadvantages. Among these are the difficulty in handling heavy steel plates at the installation site, the possibility of corrosion at the steel/adhesive interface, and the problem of obtaining clean butt joints between the relatively short steel plates [2]. These difficulties prompted the EMPA group to investigate the possibility of replacing steel plates with lightweight carbon fibre laminates. Others (e.g., [3,23-25]) have subsequently taken interest in this promising technique.

The load-deflection behaviour typically observed in tests carried out in our laboratory on reinforced concrete beams strengthened with unidirectional carbon/epoxy laminates [25] is depicted in Fig. 3. The beam dimensions in these reduced scale bending tests were 200 mm in width, 300 mm in depth, and 3000 mm long. The lower curve in Fig.3 corresponds to a concrete beam having only conventional steel reinforcement, corresponding to the minimum specified by the Canadian reinforced concrete structures code CSA A23.3. The upper curve in this figure represents the response of a beam with similar steel reinforcement, but with additional bonded laminate reinforcement on the tension face. As seen here, an appreciable strength increase can be gained by bonding a composite laminate. However, this is generally accompanied by a loss of ductility in the sense that the deflections at failure are reduced somewhat.

The load-deflection curve of a beam strengthened in bending using bonded composite plates is typically of the form ABCD seen in Fig. 3. Here point B signals the initiation of cracks in the concrete, point C is associated with the yielding of the steel reinforcing bars, and point D corresponds to the final failure of the specimen. Since the laminate reinforcement tends to reduce the overall beam deflections at failure, careful attention must be taken in the overall design to ensure that these deformations are large enough and preceded by sufficient concrete cracking to provide adequate warning of impending failure.

Various failure modes can occur when bonded composite laminates are used to increase the flexural strength of concrete beams. These include tensile failure of the bonded plate, concrete failure in the compressive zone, and sudden or continuous peeling-off of the laminate [2]. The failure mode observed in our tests was invariably a sudden separation of the laminate from the concrete beam, either at the ends of the plate or below the sections where the point loads were applied. Specially designed anchorages to delay or circumvent this type of failure would undoubtedly greatly improve this retrofitting technique.

In addition to being used for flexural strengthening, retrofitting with composite laminates has also been proposed to accommodate shear deficiencies in concrete beams [26,27]. Different shear repair schemes have been examined such as using either strips or plates bonded to the sides of the beams, as well as wrapping U-shaped laminates continuously around the sides and bottom faces. Test results show that increases in shear capacity are possible with this repair technique. The failure mode, however, is strongly dependent on the details of the bonding scheme [27] and anchorage method [26]. Investigations so far on shear reinforcement using composite laminates have been rather limited in scope and further research is clearly needed to thoroughly explore the advantages of this rehabilitation scheme.

4.0 REHABILITATION USING COMPOSITES FOR EXTERNAL PRESTRESSING

As implied by the above discussion, the strengthening and rehabilitation of reinforced concrete structures with fibre reinforced composites has for the most part been accomplished using wraps or laminates. An alternative, however, for the flexural strengthening of concrete beams is external prestressing using composite cables. Such cables have already been used for prestressing in a variety of new constructions.

An experimental study [28] where external post-tensioning was applied to previously damaged beams has confirmed that beams can indeed be upgraded and strengthened with this method. Fatigue tests up to two million cycles were also conducted as part of this study, at load levels of one-third the static strength of the beams and cable forces of 34% of their static strength. The fatigue behaviour proved to be satisfactory, resulting in negligible changes in the rigidity of the beams. The residual strengths of the beams after cyclic loading were virtually the same as the initial static strengths of the beams.

Although this rehabilitation technique has received little attention to date, it does offer interesting possibilities and no doubt merits additional consideration.

5. FIELD APPLICATIONS

Rehabilitation methods using fibre reinforced composite materials have evolved to the stage where numerous field applications have already been carried out. A large number of earthquake damaged chimneys in Japan, for example, have been retrofitted using carbon fibre winding [14]. Bonded ACM laminates have also been employed there to rehabilitate concrete highway bridge slabs as well as apartment floors [14,24].

A celebrated field application of flexural strengthening using composite laminates is the Ibach Bridge repair in Switzerland [2,21,22]. In this case, carbon/epoxy plates weighing 6.2 kg were selected for the repair instead of using steel plates since the latter would have weighed about 175 kg. Easy handling and installation, and the accompanying savings in labour costs, were obvious advantages in retrofitting with fibre composite plates. Following the successful Ibach Bridge rehabilitation, wooden bridges and other historic structures in Switzerland have been retrofitted using carbon fibre laminates [21,22].

In the U.S.A., composites have been used primarily for the rehabilitation of reinforced concrete columns. More than one hundred bridge columns, both round and rectangular, have been retrofitted using composite wraps in a number of states such as California, Nevada, Washington, Pennsylvania and Wisconsin [18]. In addition to seismic retrofitting, fibre reinforced composites have also been used to wrap cracked or spalled columns in order to provide protection against further corrosion or deterioration. Prestressed concrete water tanks, parking structure columns, light pole foundations, and structural columns in buildings have also been retrofitted using composite wraps. A striking example of the effectiveness of this seismic retrofitting technique is the performance of the Hotel Nikko in Beverly Hills, California during and subsequent to the Northridge Earthquake of January 1994 [18]. Many of the load bearing retrofitted columns of this structure were subjected to the full force of the earthquake and the considerable aftershock activity. These columns withstood the seismic loads of this significant earthquake without experiencing any damage.

6.0 RESEARCH NEEDS FOR REINFORCED CONCRETE FRAMES

The above review clearly shows that fibre reinforced composites have been extensively investigated for retrofitting concrete columns and beams, and that these materials have already been successfully applied in the field for a variety of applications. Some areas, however, have been less extensively studied than others and merit further research. Among these are the following topics: reliability and quality control, durability, long-term behaviour, fatigue behaviour, anchorage systems to prevent or reduce delamination failures, field monitoring methods, and the development of appropriate design codes.

There are apparently no reported uses of fibre composites for the strengthening and rehabilitation of concrete frames. This area is thus wide open and offers a range of topics which require research if composites are to be effectively used for the seismic rehabilitation of lightly reinforced concrete frames. Many of the results obtained for column and beam rehabilitation using composites can obviously be carried over to frame applications. However, special issues will no doubt arise related to joints and base details, and research on these aspects will likely be required.

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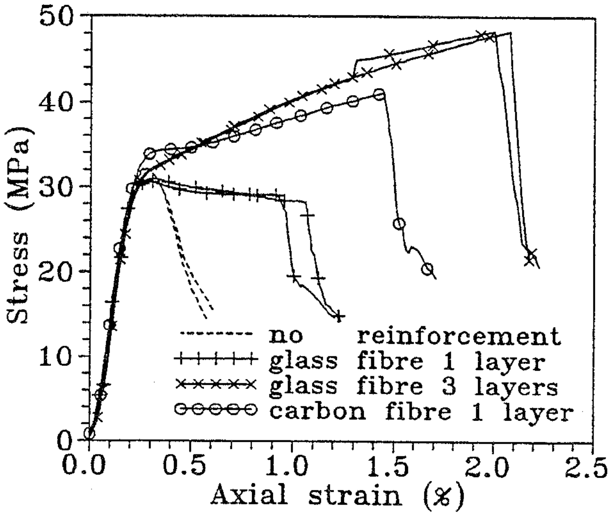
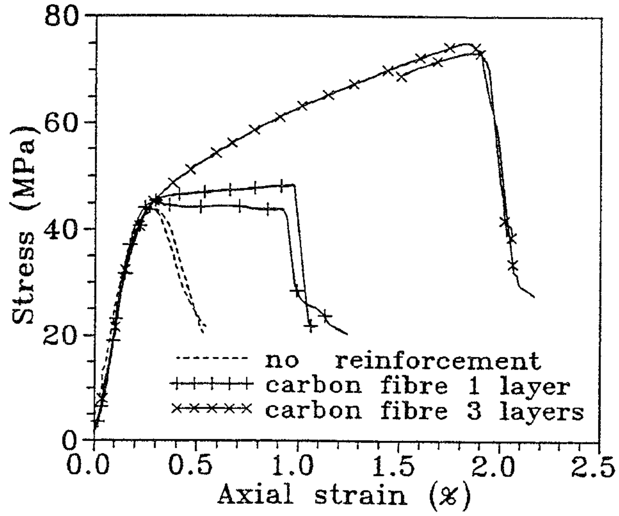
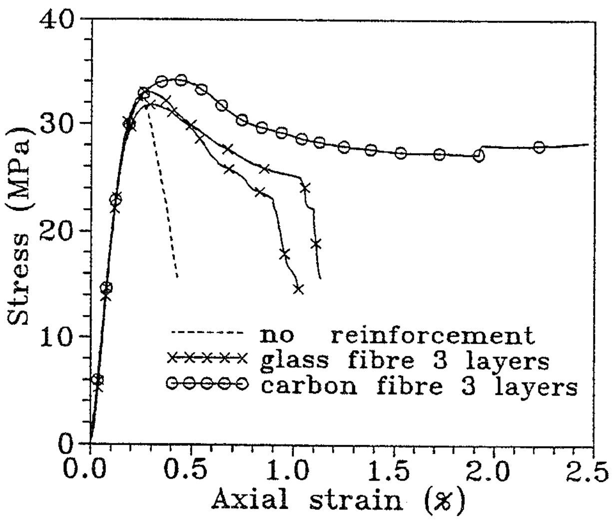


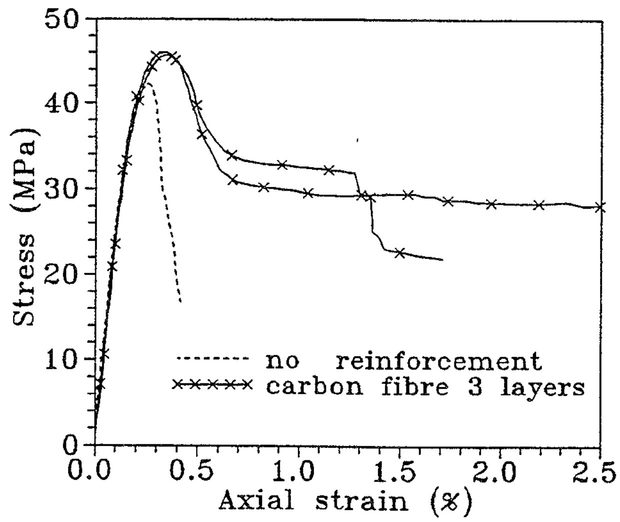
(a) Round column.



(b) Round column.



(c) Square column.



(d) Square column.

Fig. 1: Behaviour of composite wrapped concrete columns

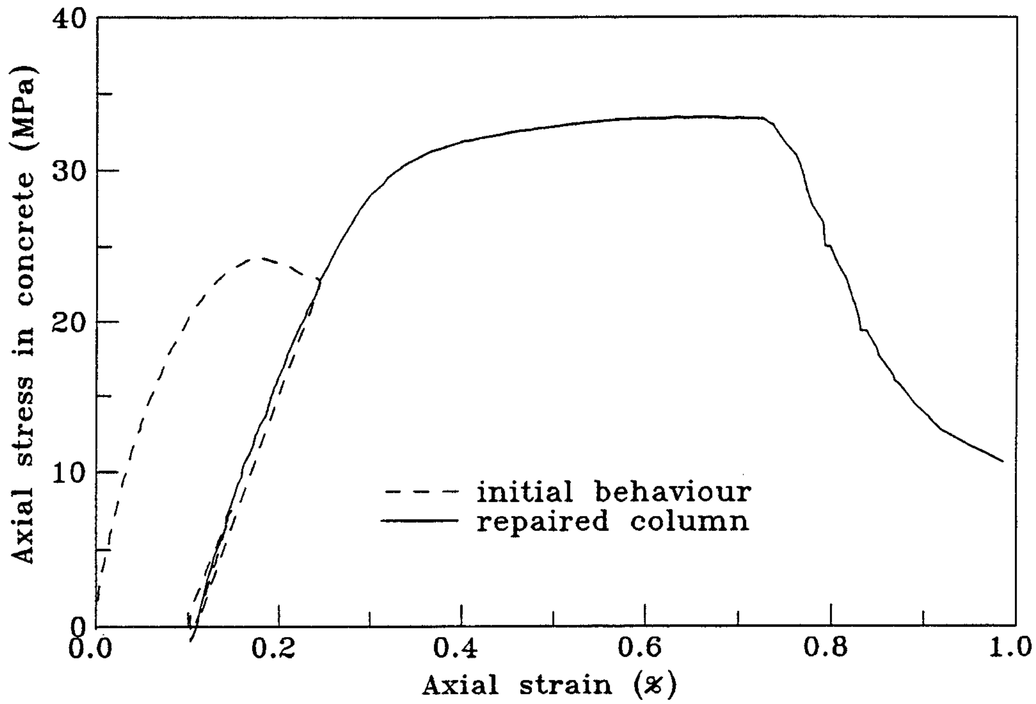


Fig. 2: Repair of reinforced concrete column with composite wrap

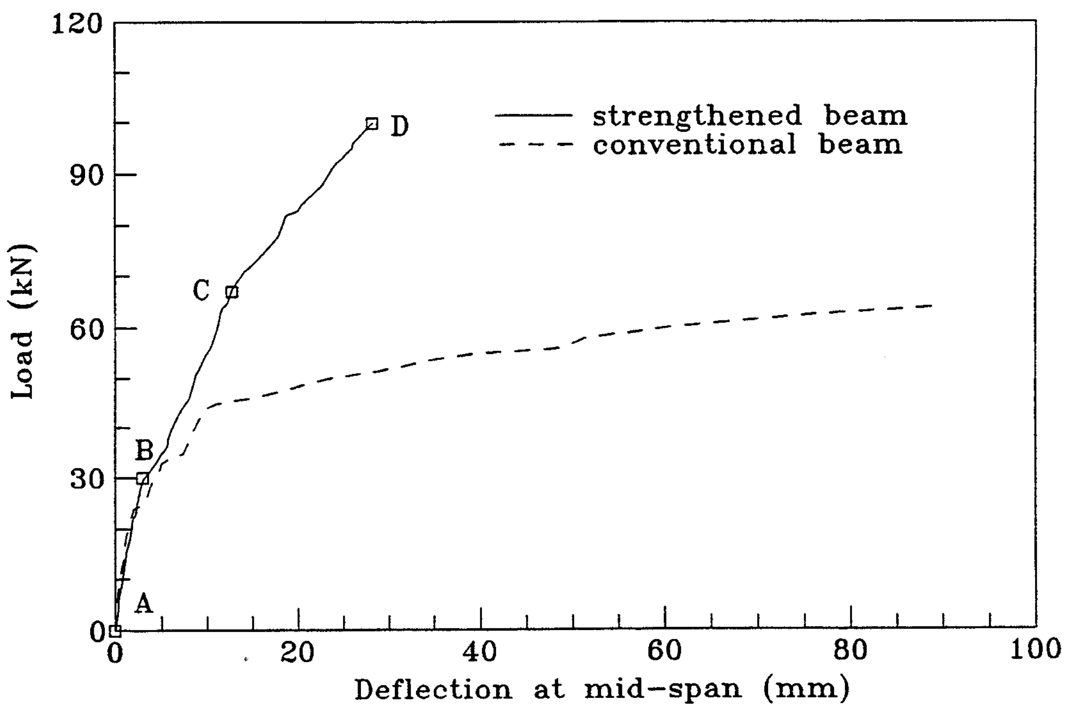


Fig. 3: Strengthening of reinforced concrete beam with composite laminate

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