State of Research on Seismic Retrofit of Concrete Building Structures in the US

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Introduction

The majority of buildings in regions of high seismicity in the United States do not meet current seismic code requirements, and many of these buildings are vulnerable to damage and collapse in an earthquake. Concerns for seismic rehabilitation of existing buildings grew considerably following the 1971 San Fernando earthquake and resulted in several programs to identify and mitigate seismic risks. The 1989 Loma Prieta and 1994 Northridge earthquakes provided significant new impetus for seismic rehabilitation of buildings in California and elsewhere in the US. Earthquakes in other parts of the world provide a continual reminder of the need for seismic mitigation programs underpinned by research to demonstrate their effectiveness and improve the efficiency.

Seismic rehabilitation research in the US includes individual investigator and coordinated program research efforts. The US National Science Foundation began to fund research on seismic rehabilitation in earnest in the early 1980s. The early efforts were not overtly coordinated, and it became apparent that these programs would be unlikely to comprehensively address the broad needs in terms of range of construction and performance objectives necessary for the development of research-based consensus design guidelines. In 1990, the National Science Foundation announced a five-year coordinated research program on seismic repair and rehabilitation of buildings. The objectives of the program were to provide information for evaluation of the vulnerability of existing structures for various levels of seismicity, and to develop economical construction techniques for repairing and strengthening hazardous structures. The program culminated with the publication of a special theme issue of Earthquake Spectra [Earthquake Spectra; 1996]. The NSF research effort was supplemented by research carried out at the National Center for Earthquake Engineering Research [e.g., Beres; 1996].

In the 1990s the Federal Emergency Management Agency (FEMA) and the State of California separately began to develop seismic rehabilitation guidelines. These efforts were guided by research reported to date or under way at the time. For the FEMA effort, the American Society of Civil Engineers subcontracted a research synthesis project resulting in a compilation of previous research in electronic format. The California effort resulted in a research synthesis specific to concrete buildings [Moehle, 1994]. Applications of the FEMA 273 Guidelines [FEMA, 1997] and California-directed ATC 40 Guidelines [ATC, 1996] to rehabilitation projects has revealed additional research needs, several of which are being addressed by ongoing research [e.g., Moehle, 2000]. The symbiosis between researcher and practitioner is leading to rapid advances in the state of the art in seismic rehabilitation the US.

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

Typical Configurations and Details

The development of details suitable for seismic resistance of concrete buildings was a gradual process in the US, and continues today. Main advances in understanding were made in the 1960s with the publication of the text by Blume, et al. [Blume, 1961]. While publication of this text along with 1960s and 1970s editions of the Structural Engineers Association of California (SEAOC) Blue Book resulted in some improvements to building design practices, it was not until the 1976 Uniform Building Code [UBC, 1976] that ductile detailing practices became mandated in the western US. The specified details were surprisingly similar to those of today. Buildings constructed prior to that time commonly have significant deficiencies in configuration and detailing.

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

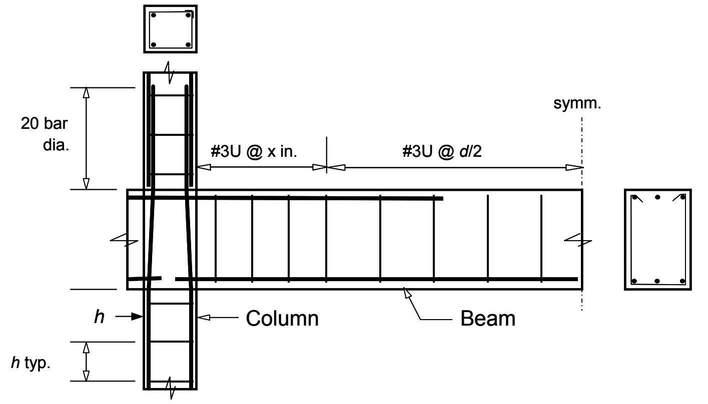
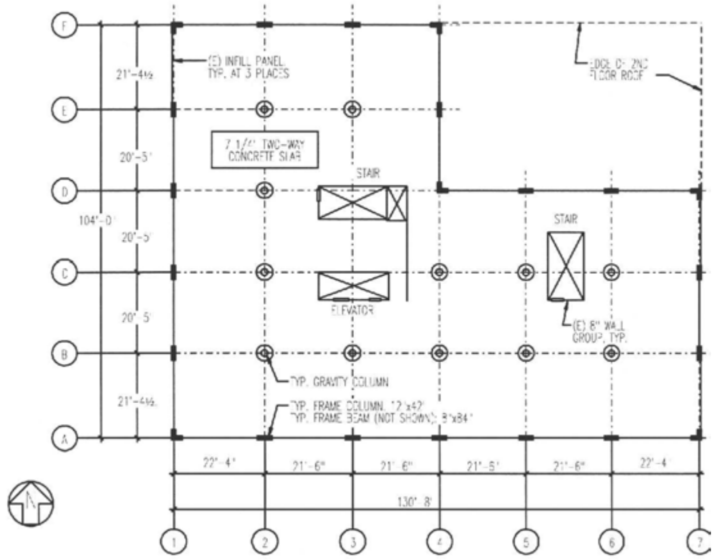


Figure 1 – Typical details before 1976 UBC

Most existing concrete buildings in the highly seismic western US comprise a mix of beam-column frames and shear walls; frame buildings are not typical. Figure 2 is a floor plan of a representative building showing beam-column and shear wall framing. While the walls may provide most of the lateral resistance, not insignificant resistance may arise from the beam-column frame. Current practice usually aims to include the contribution of the beam-column frame so that rehabilitation is minimized. Whether this is the case or not, current practice requires that the beam-column frame be demonstrated to sustain gravity loads without collapse for design-level events.



Performance in Past Earthquakes

The most significant failures of reinforced concrete buildings in past earthquakes have been attributed to column failures. Causes have included column shear distress, spalling of column end regions, buckling of column longitudinal reinforcement, and formation of soft stories. Several collapse of one or more stories of buildings have been attributed to column failures (e.g., Figure 3).

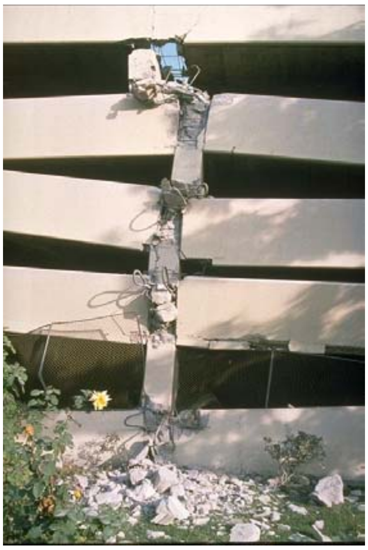


Figure 3 – Column failures – Northridge earthquake

Figure 4 – Joint failures – Northridge earthquake

Failures of beam-column connections also have been observed. Figure 4 depicts an example from the Northridge earthquake.

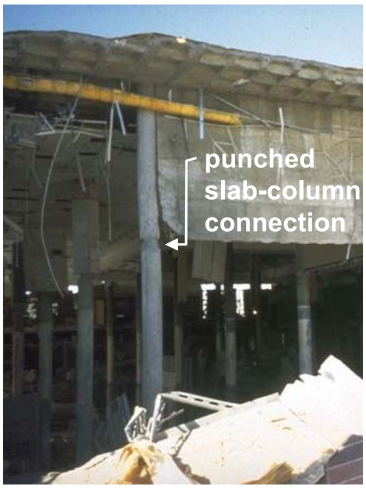


Figure 5 – Slab-column connection failure – Northridge earthquake

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

Damage to shear walls and to coupling beams, while costly and disruptive, generally have not resulted in building collapse, and therefore have received less attention than have columns, joints, and slab-column connections.

Failures in structures, while sometimes attributable to specific details, often have more systemic causes. Attachment of architectural elements, such as the parapet walls in the parking structure of Figure 3, can increase stiffness of components in specific locations of a building resulting in overload and premature failure. Weak-column/strong-beam systems are prone to story failures, especially in frames having columns with widely-spaced ties. Excessive flexibility in frames, as well as in frame-wall structures with flexible foundations may result in failure of framing components owing to excessive drift. The dividing line between damage without collapse (Figure 6) and damage with collapse has not been identified analytically.

Rehabilitation Approaches

Two general approaches usually are considered for a seismic rehabilitation project in the US. The first, illustrated in Figure 7, involves global modification of the structural system. In this approach, the modifications to the structural system are designed so that the design demands, often denoted by target displacement, on the existing structural and nonstructural components are less than their capacities. Common approaches include addition of structural walls, steel braces, or base isolators. Passive energy dissipation schemes are not common for reinforced concrete frames because the displacements required for them to be effective often are beyond the displacement capacities of the existing components. Active control is rarely used.

Another approach, illustrated in Figure 8, involves local modification of isolated components of the structural and nonstructural system. In this approach, the objective is to increase the deformation capacity of deficient components so that they will not reach their specified limit state as the building responds at the design level. Common approaches include addition of concrete, steel, or fiber reinforced polymer composite (FRPC) jackets.

Recent experience in the US is that global modification schemes are more common than local modification schemes. However, difficulties in developing accurate models of foundation flexibility and (believed-to-be-over-) conservative acceptance criteria for existing components requires use of some combination of the two approaches.

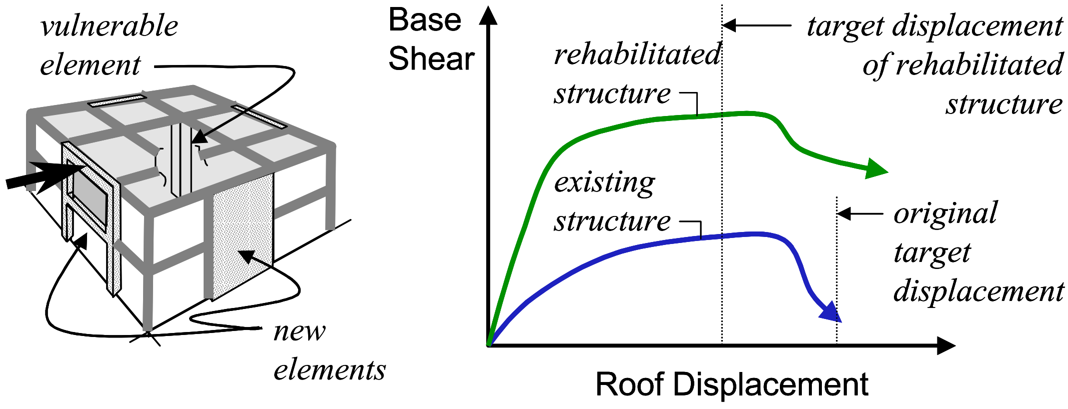


Figure 7 – Global modification of the structural system

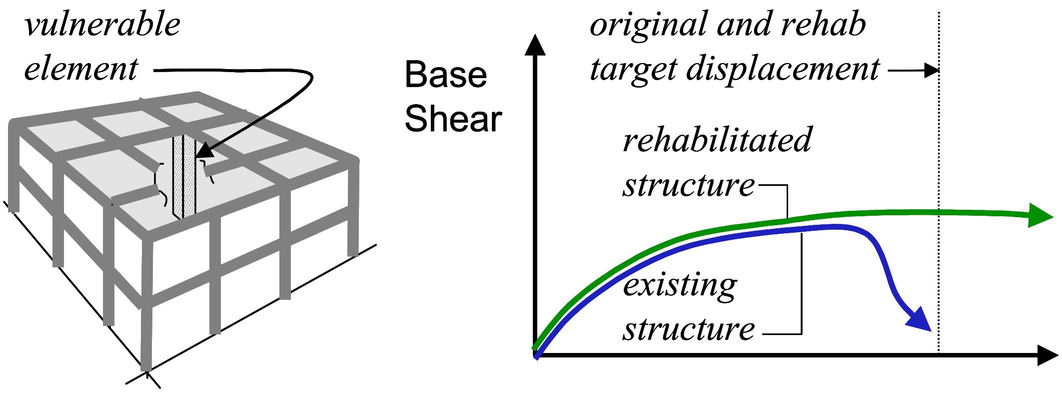


Figure 8 – Local modification of structural components

Research on Upgrading Global Deficiencies and Evaluating Capacities of Existing Construction

Adding Elements to Reduce Drift and Decrease Ductility Demand -

Addition of new reinforced concrete walls is a common method of seismic rehabilitation. The objectives usually are to reduce lateral drifts overall, as well as to avoid story mechanisms. In design, attention must be paid to distribution of the walls in plan and elevation (to achieve a regular building configuration); transfer of inertial forces to the walls through floor diaphragms, struts, and collectors; integration and connection of the wall into the existing frame; and transfer of loads to the foundation.

Jirsa and Kreger [Jirsa, 1989] have reported tests on one-story infill walls. Four specimens were tested. The first three varied in opening location: full infill, door opening, and window opening. Although the infill increased the strength of the frame, failure was relatively brittle owing to failure of the existing column lap splices. In a fourth specimen, new longitudinal reinforcement was placed adjacent to the existing columns to improve continuity of the tension chord of the wall. Strength and ductility were notably improved (Figure 9).

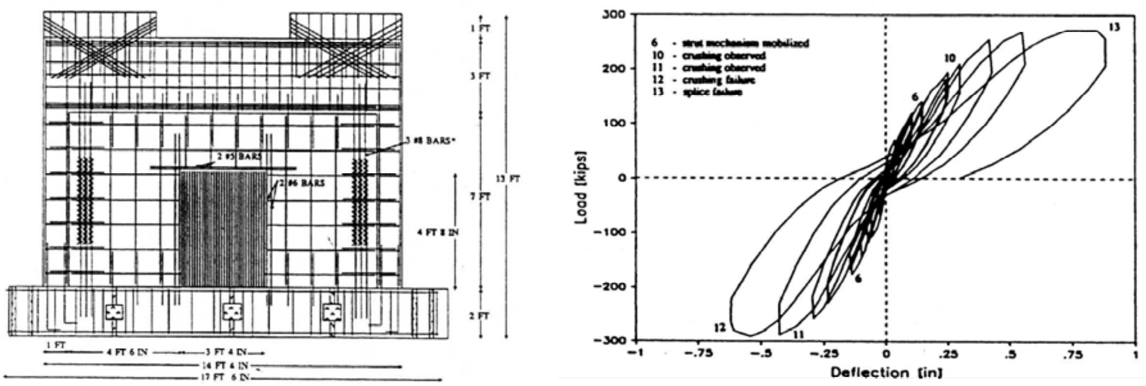


Figure 9 – Infill wall. Note the three No8 vertical bars added as boundary reinforcement.

Bush et al. [Bush, 1990] have investigated use of wing walls as a retrofit measure for an existing frame with weak columns and deep spandrel beams. The concrete piers extended the full height of the frame. To develop monolithic behavior between the original frame and the strengthening members, shear friction and dowel action were employed. Existing surfaces were sandblasted, and lug action was also engaged by casting new concrete into window space. The system was designed so that beams would yield in flexure when the piers were at only 40 percent of their shear capacity. The system showed ductile response during testing (Figure 10).

Frosch et al. [Frosch, 1996] investigated use of precast panels for frame infills, the objective being to reduce disruption to building function during construction. The panels with shear keys were sized to be moveable within the building using a forklift and elevator. Shear transfer was through steel pipe dowels cast through the floor beams, which also provided access for casting and additional reinforcement. Post-tension steel was provided for moment strength because of the inadequacy of existing column longitudinal reinforcement splices. Walls were tested successfully in flexure tests and shear tests. Results for the shear test are shown (Figure 11).

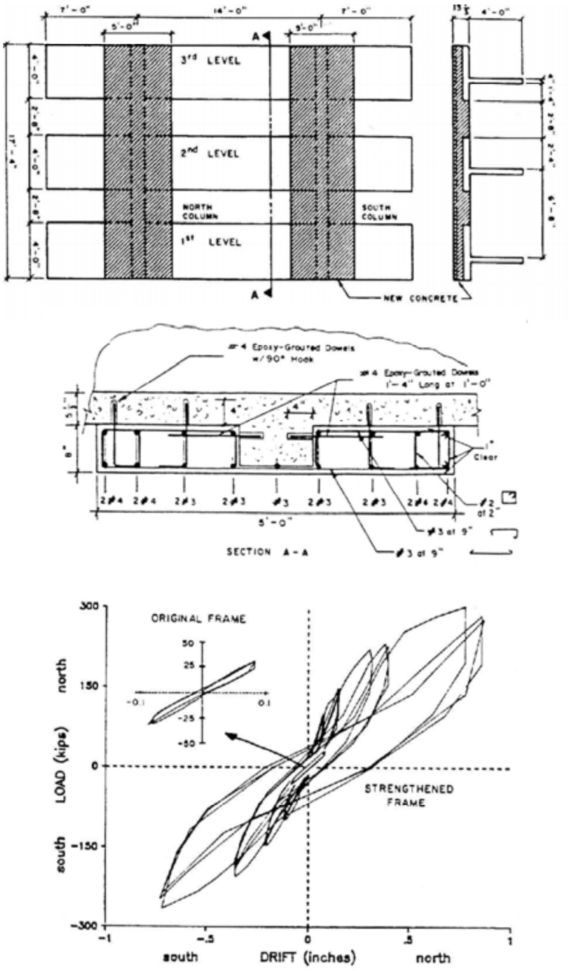


Figure 10 – Wing-wall retrofit. Note: new concrete dowelled to existing concrete.

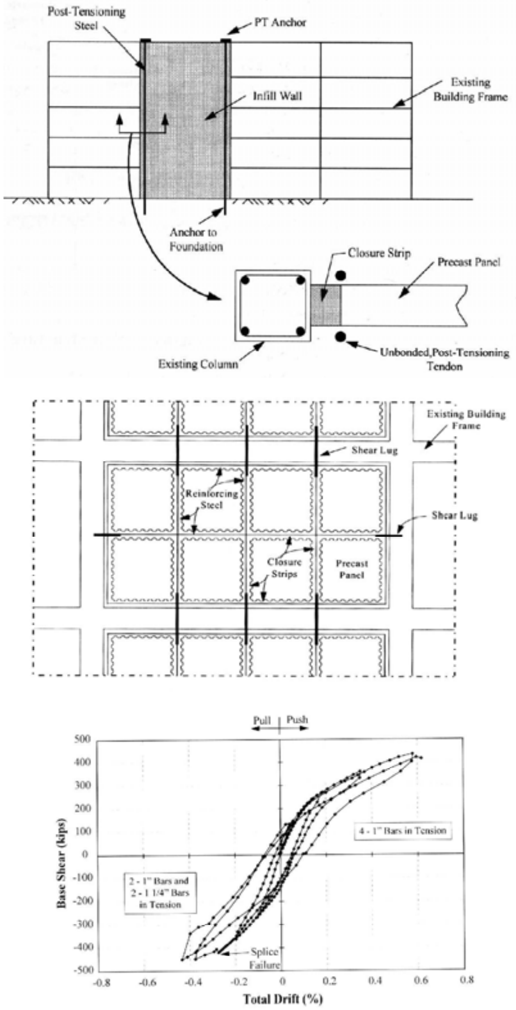


Figure 11 – Precast wall retrofit. Note PT for boundary reinforcement.

An important element in the design of wall retrofits is the integration of the new materials with the existing materials. In some cases, lug action between new concrete and existing concrete can be counted on, as in the case where concrete is infilled within windows (Figure 10). Studies of shear transfer have been reported by Bass et al. [Bass, 1989] and Luke et al. [Luke, 1985]. Bass et al. found that shear strength increased with increasing interface reinforcement, embedment depth, and concrete strength. Interface surface preparation did not show a definite effect. Use of a bonding agent was not justified by the results; vertically and horizontally cast surfaces behaved similarly, and overhead casting with drypack resulted in reduced strength.

Behavior of existing column lap splices is important when the columns become boundary elements of infilled walls, as noted in the previous studies. Techniques for rehabilitating existing laps include removing cover concrete and welding overlapped bars, confining lap splices by steel or reinforced concrete jackets, and providing new reinforcement in a jacket. Valuvan et al. [Valuvan, 1993] reports results on in-situ column laps retrofitted by welding, steel angles, and external and internal hoops. Behavior of the specimens retrofitted by steel elements was varied, as it was difficult to match the steel elements with the existing concrete surface. The specimens with steel ties also exhibited varied performance; grouting the ties proved to be essential to good behavior. Behavior of the specimens with internal ties was unsatisfactory, possibly because chipping the cover concrete to place the new ties resulted in irreparable internal damage. Figure 12 summarizes some of the results.

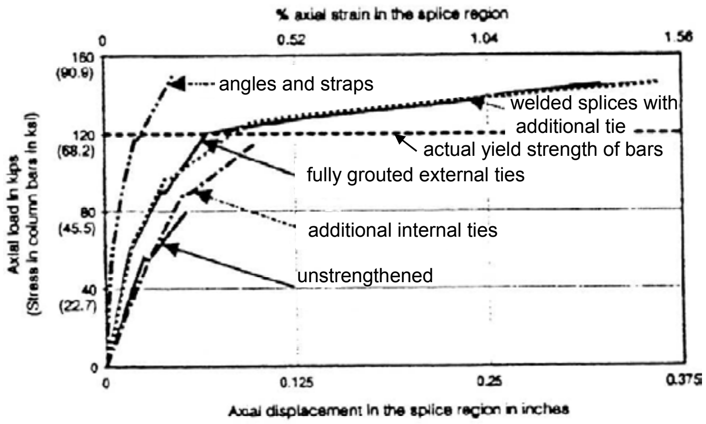


Figure 12 – Behavior of retrofit column lap splices

A range of steel bracing systems have been proposed for upgrading existing concrete frames. These include concentric and eccentric bracing systems, as well as post-tensioned bracing systems. Several studies of steel bracing systems have been reported in the US [Badoux, 1990; Bouadi, 1993; Bush, 1991; Goel, 1990; Masri, 1996]. In some studies, the steel is fit within the concrete frame, in which case continuity must be provided by steel passing through the floor system. In other studies, the steel frame is intended to be attached to the perimeter of the building, in which case the framing can be continuous over height. Examples of the two cases are in Figure 13. Collectors are needed to transfer floor loads into the new framing. These studies have shown that total strength of the retrofit system can be determined as the composite strength of the steel/concrete system; direct summation of the concrete frame and steel frame strengths often will underestimate the total strength.

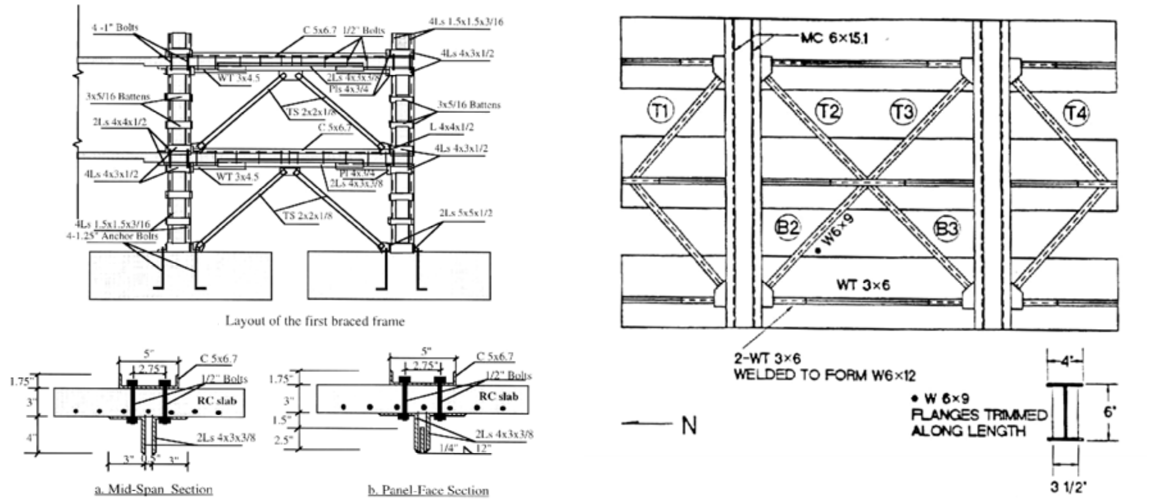


Figure 13 – Steel bracing retrofits.
(Left figure from Masri, 1996. Right figure from Bush, 1991.)

Evaluating Capacity of Existing Construction –

Design of a global retrofit system should aim to control demands on the existing framing system so that they do not exceed capacities (Figure 7). Capacity of existing concrete construction, therefore, has become a focus of research. Of primary interest has been response of existing columns, beam-column joints, and slab-column connections.

Research on reinforced concrete columns aims to understand behavior in flexure, axial load, shear, and bond (especially as this relates to failure of lap splices). Studies by Lynn et al. [Lynn, 1996] and Moehle [Moehle, 1999] have examined shear strength, lap-splice strength, and deformation capacity as limited by loss of axial load. As shown in Figure 14, axial load capacity often is maintained to deformations well beyond the point where lateral load capacity is exhausted – most reported experiments do not clearly identify where axial load failure occurs. Deformation capacity at collapse depends on the failure mechanism (flexure versus shear) and the level of axial load.

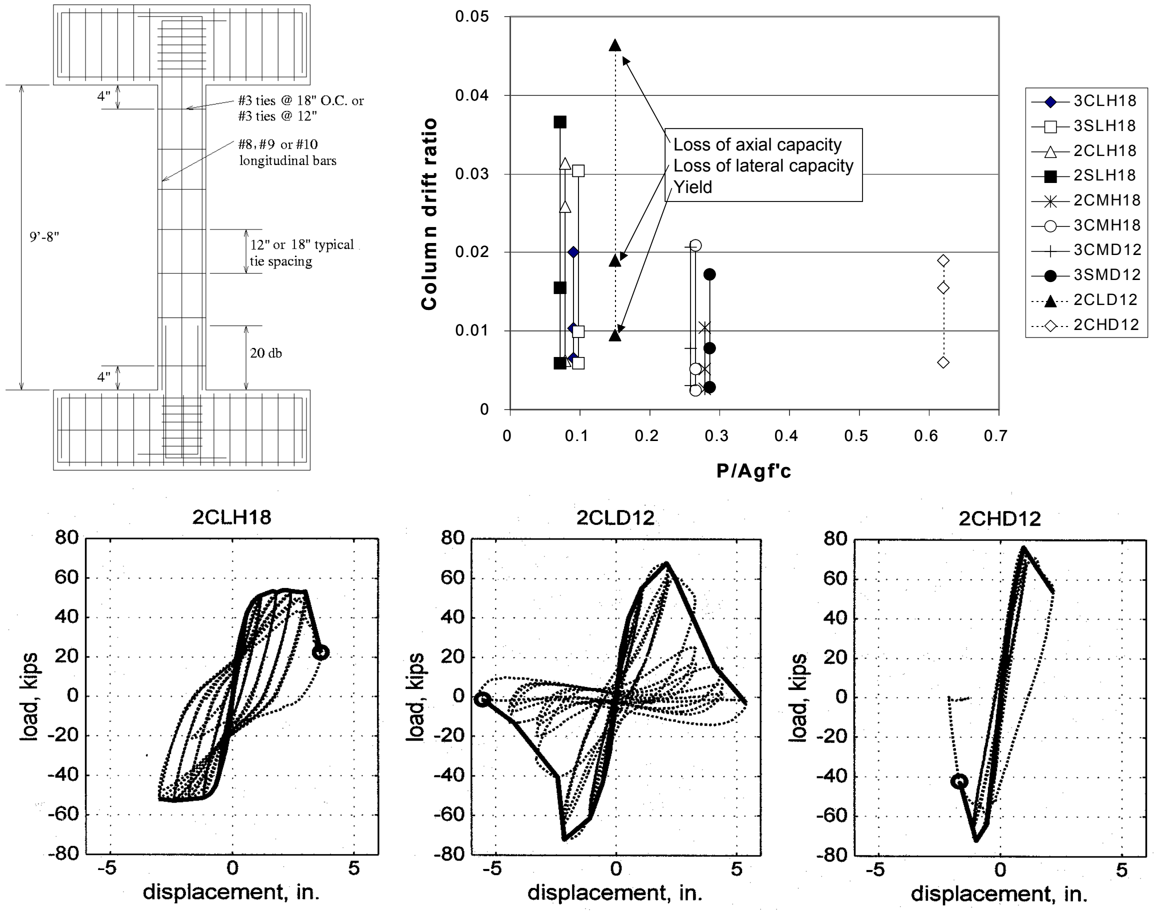


Figure 14 – Deformation capacity of existing columns.

Similar studies are underway to understand the strength and deformation capacity of existing joints (Pantelides at the University of Utah and Lehman at the University of Washington). As with columns, these studies are identifying influence of load history and construction details on the failure mechanisms. The studies benefit from other work on lightly-reinforced connections, notably that of Beres et al. [Beres, 1996]. That work identified effects of joint configuration and axial load on joint strength (Figure 15). Further work is needed to understand deformation capacities and effects of joint failures on loss of axial load carrying capacity.

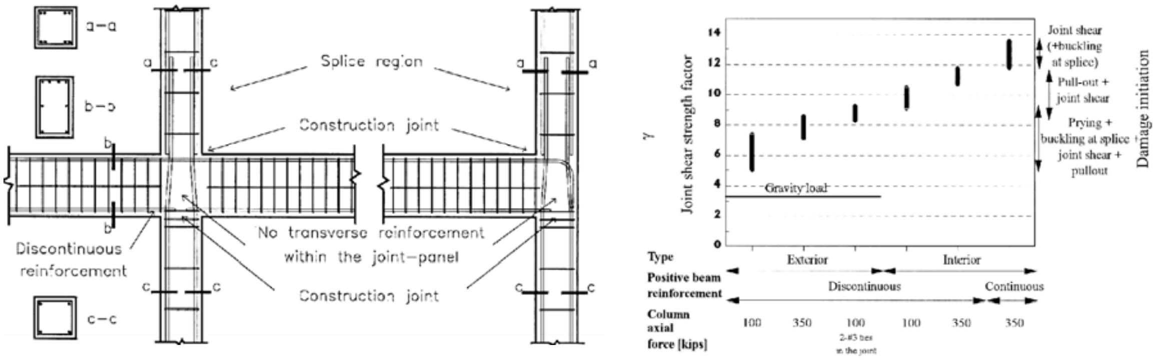


Figure 15 – Strength of existing joints.

Pan and Moehle [Pan, 1989] reported on deformation capacity of reinforced concrete flat plate construction, identifying a simple rule relating deformation capacity and gravity shear level for typical construction (Figure 15). Martinez, et al. [Martinez, 1994] extended this work to post-tensioned flat-plate construction. Studies of details consistent with older construction have been reported by Dovich and Wight [Dovich, 1996] and Durrani???

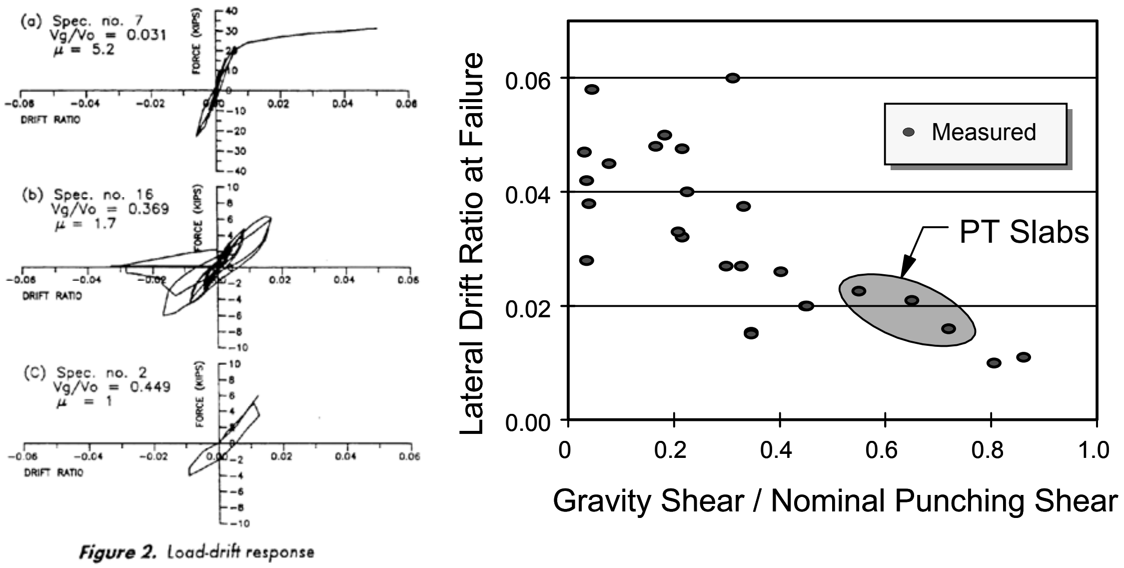
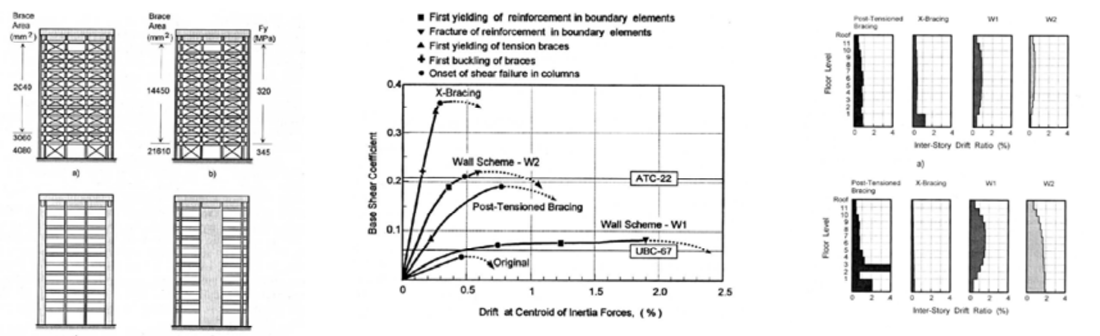


Figure 16 – Deformation capacity of flat-plate construction

Analytical studies have provided insights into the efficacy of various global retrofit schemes. For example, Pincheira and Jirsa [Pincheira, 1995] used inelastic static and dynamic analysis of three prototype buildings. Retrofit schemes included the post-tensioned bracing, structural steel bracing, and infill walls of reinforced concrete. The studies show that there is no unique solution and that several difference retrofit schemes can be designed to provide effective seismic resistance. Satisfactory response was obtained only for schemes that adequately controlled interstory lateral drifts. This study, and others, identify global retrofitting strategies and simplified design methodologies as important areas for continued research.



Retrofit schemes: a) Post-tensioned, b) X-braced, c) wall W1, d) wall W2

Comparison of calculated relations between base shear and roof displacement

Distribution over height of story drift for different records and retrofit schemes calculated by inelastic dynamic analysis

Figure 17 – Analytical studies of different retrofit schemes

Research on Upgrading Local Deficiencies

Research in the US on upgrading local deficiencies lately has concentrated on columns, beam-column joints, and slab-column joints. Beams and walls generally are not considered as critical in most US buildings, and therefore have not been primary research subjects. Foundations, while viewed by some as being important as part of the seismic retrofit, have not been much studied because of the difficulties of in-situ testing.

Columns -

Response of a column in a building frame may be controlled by combined axial load, flexure, shear, and anchorage/bond. It is widely accepted that flexure should be the controlling mechanism for a column, although for some details and proportions in existing buildings the flexural deformation capacity may be inadequate. It also is widely accepted that columns should not be the weak link in the building frame unless full-height walls are present to control story mechanisms; thus it may be necessary to increase strength in flexure, shear, and anchorage so that the column remains essentially elastic.

A variety of approaches have been used to improve the flexural response of columns. The technique employed depends on the desired objective. Where the objective is to increase column flexural strength, approaches include new longitudinal reinforcement passing through the floor system and encased in a concrete jacket; and steel sections placed alongside and acting compositely with the columns. Alcocer [Alcocer, 1993] tested conditions with distributed and bundled longitudinal reinforcement passing through the floor system (Figure 18). The retrofit succeeded in moving plastic hinges from the columns to

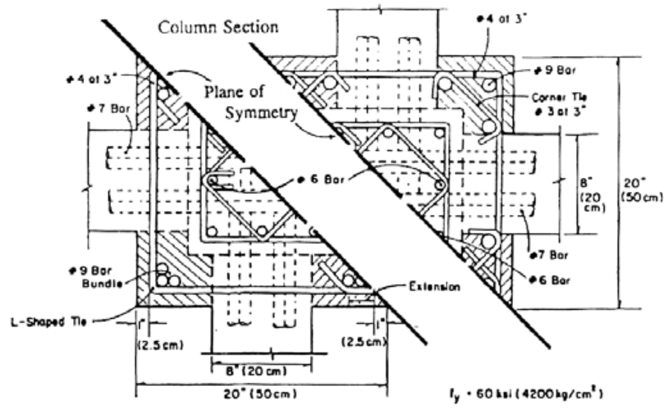


Figure 18 – Retrofit to add column flexural strength

the beams. Goel [Goel 1990] has used steel angle sections to act compositely with the columns, thereby increasing their flexural strength. Prestressing steel also has been suggested as a retrofit technique to increase flexural strength [Choudhuri, 1992].

Inadequate spacing and configuration of transverse reinforcement, resulting in inadequate concrete confinement and a propensity for longitudinal reinforcement buckling, often limit flexural response of existing columns. To increase flexural deformability, a variety of approaches have been studied, including encasement by steel or concrete jackets, confinement by welded wire fabric, confinement by added ties, and other approaches. So that flexural strength is not increased, a gap commonly is left between the added material and the end of the column. Choudhuri [Choudhuri, 1992] reported on cast-in-place concrete jackets for small-scale columns. Stoppenhagen [Stoppenhagen, 1987] reported results from tests of a two-story, strong-spandrel frame retrofitted by encasing the columns in reinforced concrete jackets. The columns sustained damage during testing, but overall performance of the retrofitted frame was excellent (Figure 19).

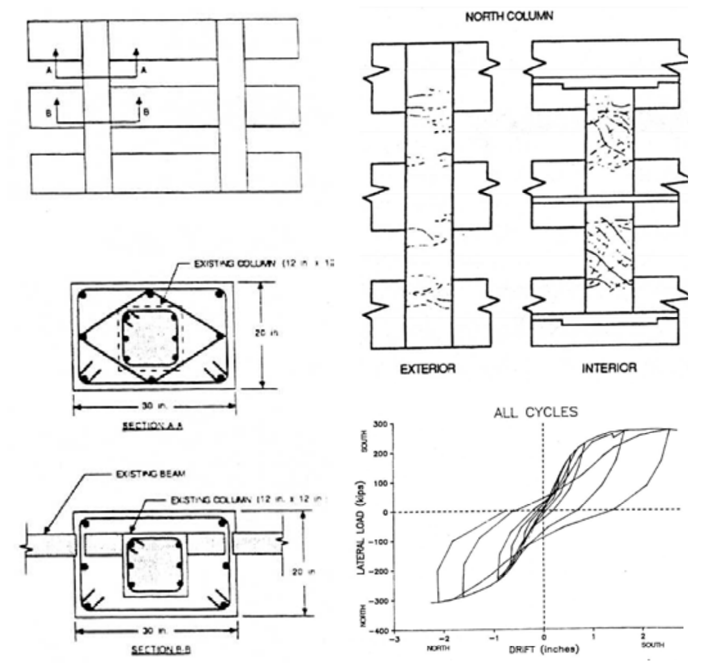


Figure 19 – Concrete encasement of columns

More recent work has emphasized applications of composites. Harries et al. [Harries, 1998] report on use of carbon fiber reinforced polymer composite (FRPC) jackets to retrofit non-ductile columns. Design of the jackets was based on providing sufficient confinement pressure while limiting the tensile strains in the jackets. They were effective to confine the flexural plastic hinge zone without significantly increasing strength or stiffness (Figure 20). Eventual column failure resulted from deterioration of the compression zone and longitudinal reinforcement buckling in the hinge region, suggesting that greater out-of-plane stiffness is required for the jacket.

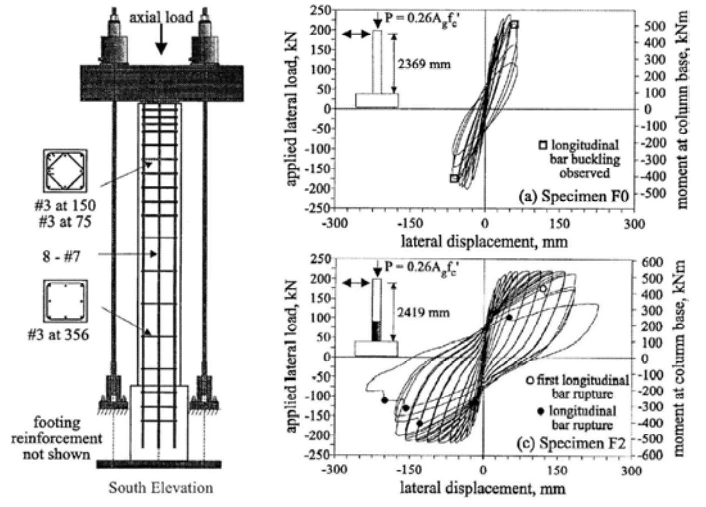


Figure 20 – Column retrofit by carbon FRPC

Lap splices of column longitudinal reinforcement can be improved by several techniques including removal of cover concrete and welding overlapped bars, confining the lap by steel or reinforced concrete jackets, and providing new longitudinal reinforcement in a jacket. Aboutaha et al. [Aboutaha, 1996] report on use of steel jackets to confine inadequate laps. The tests showed that the retrofit approach can be effective using thin steel jackets augmented by a small number of adhesive anchor bolts. The anchor bolts stiffen the thin plates, and are especially important as the column width increases. Alternative approaches were investigated by Valluvan et al. [Valluvan, 1993], as described previously (Figure 12).

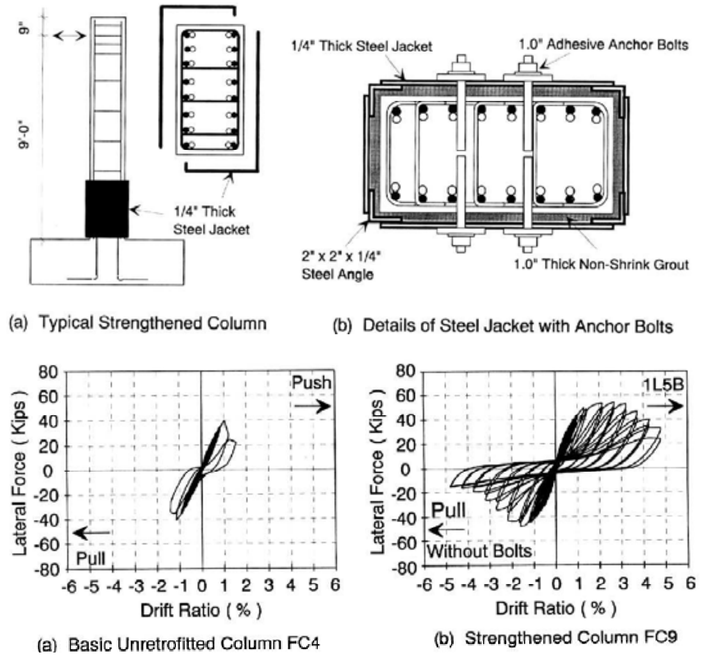


Figure 21 – Steel jacket retrofit of column laps

Shear strength can be increased by adding steel, reinforced concrete, or composite jackets. In most studies, the strengthening element is discontinued at the column end to avoid increasing the flexural strength and corresponding shear demands. Aboutaha et al. [1999] report on use of steel jackets of a variety of configurations for enhancing shear resistance. The tests showed that such columns can be effectively retrofit with thin rectangular plates (width to thickness ratios were as large as 144) (Figure 22). Other work has been reported by Bett et al. [Bett, 1988] and Stoppenhagen et al. [Stoppenhangen, 1987]

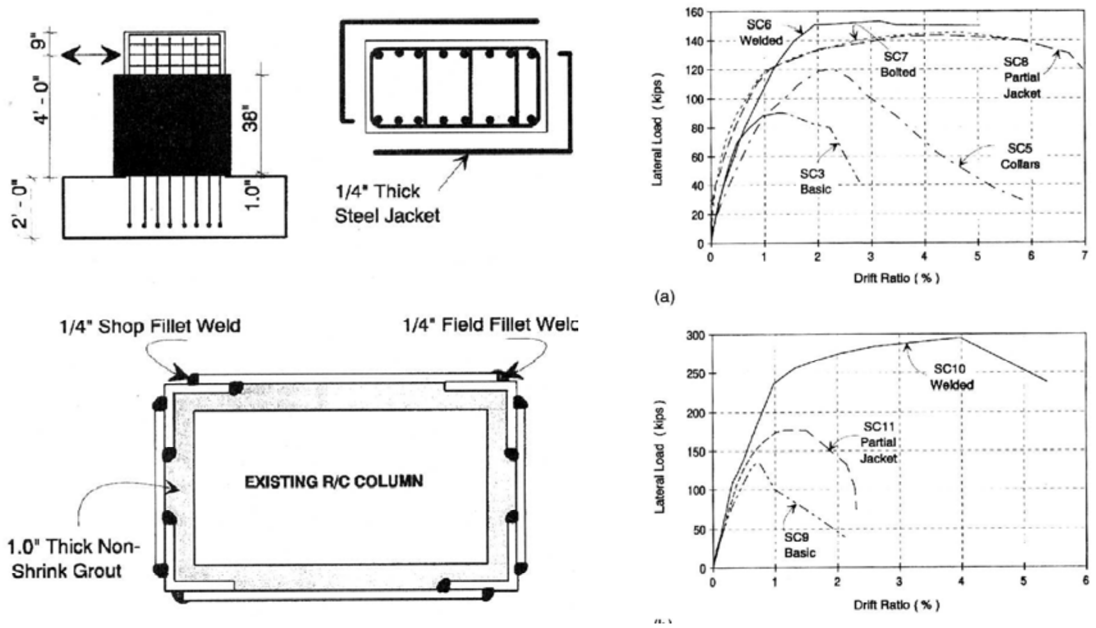


Figure 22 – Steel jacket retrofit for shear

Slab-Column Framing –

Punching shear failure of two-way slabs was discussed previously (Figure 16). The consequences of shear failure depend largely on the details of the reinforcement at the connection. Following initial punching, continuous bottom reinforcement passing through the column can suspend the slab on the column by catenary action, thereby preventing progressive collapse that can result in pancaking of an entire building. Continuous top bonded reinforcement is less effective in suspending the slab. Continuous post-tensioning reinforcement located near the top surface of the slab has been found to be effective in suspending the slab after punching failure [Martinez, 1994].

Several approaches to retrofitting deficient slab-column connections have been proposed. Masri [1996], Lou [1994], and Martinez [1994] have reported using steel or concrete drop panels added below the slab to increase the punching shear perimeter and therefore the shear strength. Martinez [1994] also presented results of tests in which the slab was retrofit using steel plates on both sides of the slab with through-bolts to act as shear reinforcement. The steel plate was epoxied to the slab. The concrete capital solution was achieved by chipping out the concrete around the column and recasting the slab with the capital. Both solutions were effective.

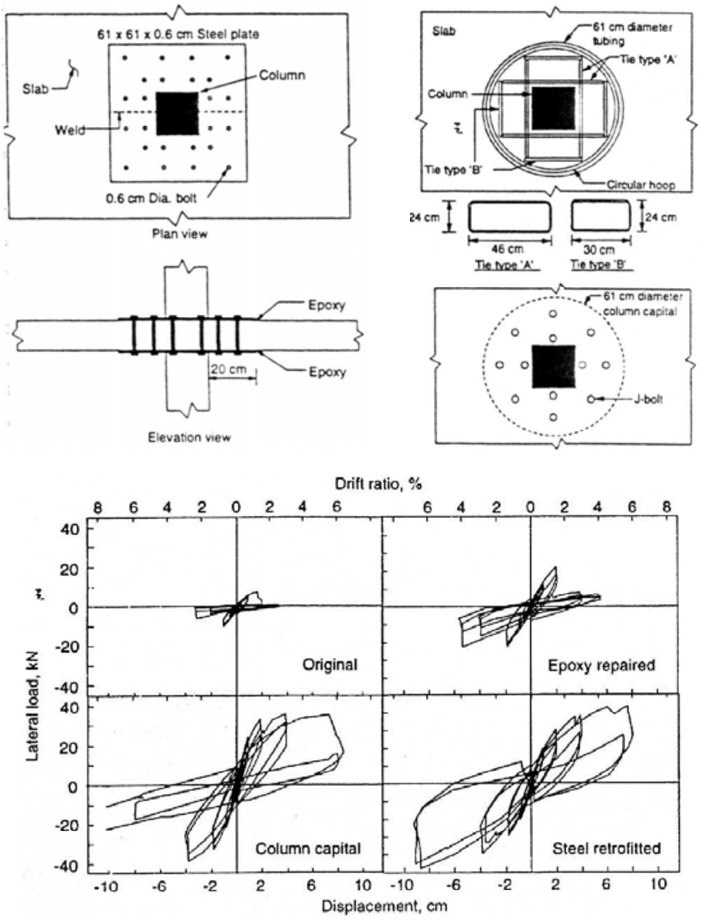


Figure 23 – Retrofit of slab-column connections

Beam-Column Joints -

Retrofitting approaches for beam-column joints include steel and reinforced concrete jackets to confine and strengthen the joint, and reinforced concrete jackets with new continuous longitudinal reinforcement to strengthen and possibly confine the joint. Alcocer [1993] reports tests of jacketed joints. In one case, the column jacket had bundled bars, in another the column had distributed bars, and in the third a column was jacketed with distributed bars while the beam also was jacketed (Figure 18). All the strengthening schemes included a joint jacket comprising a structural steel cage welded around the joint after casting the column jacket. Overall behavior of the specimens was satisfactory (Figure 24). Other joint retrofitting studies include those of Beres [1992b], Choudhuri [1992], Corazao [1989], and Krause [1990].

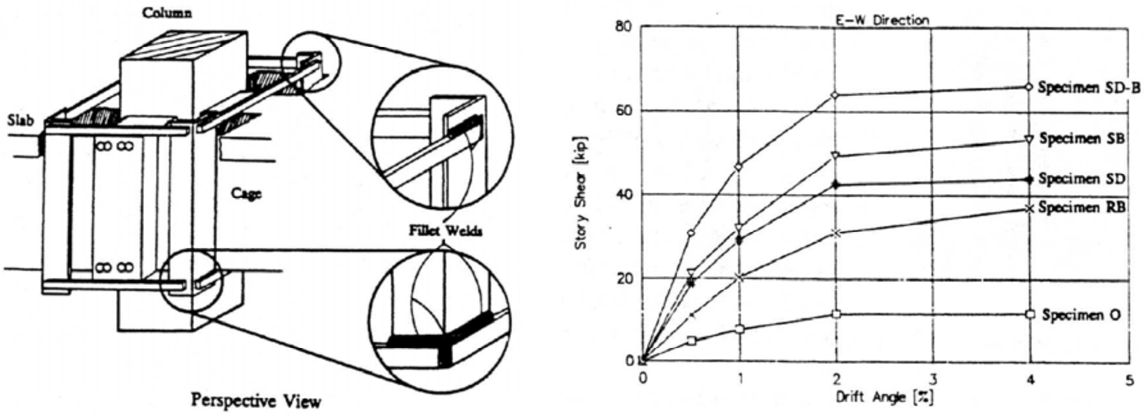


Figure 24 – Jacketed beam-column joints

Closing Remarks

The state-of-the-art of rehabilitation of reinforced concrete building structures has advanced prominently in the past two decades owing to dedicated individual and coordinated research efforts. These works have contributed not only to a growing body of knowledge on seismic retrofitting, but also have provided the underpinnings of major guidelines efforts at state and national levels. Continued application of these guidelines identifies pressing needs for research to support practical and effective application of seismic retrofitting to existing buildings. Needs are continually identified related to the technology of seismic retrofit (physical implementation and efficacy of retrofit measures) as well as methodologies for the numerical evaluation of performance of systems that have been retrofitted. International collaboration in retrofitting research is a necessary component of rapid advancement in this field.

Acknowledgments

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