SECTION 2

RETROFIT OF GLD R/C FRAME STRUCTURES

2.1 Introduction

Many gravity load designed (GLD) reinforced concrete frame structures, not specifically designed to withstand earthquakes, have survived minor, moderate, and severe magnitude earthquakes (Armenia, Turkey, Loma Prieta, and Mexico City). Their survival is because they have some inherent strength for resisting lateral forces. However, this inherent strength can not be regarded as sufficient for resisting all moderate or major type earthquakes, since earthquakes vary in magnitude, frequency content, and striking direction. In many areas, however, questions are repeatedly asked: Should a structure be retrofitted to adequately resist the large seismic forces of an earthquake? Or is the probability for occurrence of a strong ground motion too small to warrant retrofitting? The seismic retrofit (upgrade) of an undamaged structure to adequately absorb the seismic forces of an unexpected future earthquake can potentially be an expensive proposition. In the Eastern United States and other low seismicity zones, it may be very difficult to convince owners and/or government officials to invest in such retrofit of structures, except possibly for special structures. To address this dilemma, the study herein focuses primarily on relatively inexpensive retrofit techniques that can be applied to either damaged or undamaged structures in low to moderate seismicity areas. The same structural retrofit may also be required to guard against other hazards which produce large lateral loads such as hurricanes, tornados, and blasts.

2.2 Assessment of Seismic Damage States for R/C Structures

Following an earthquake, an engineering inspection and assessment of damage for most structures, including buildings, bridges, retaining walls, homes, apartment buildings, etc., may be required for further serviceability and for safety to the community. In this study, consideration will only be given to R/C frame structures and the damage which typically occurs in these structures from earthquake forces. The following classifications define damage states and limits along with a descriptive condition of a structure following an earthquake:

(a) ***minor damage - "serviceable" condition***. For this classification, the extent of damage to the structure may vary from no damage to slight cracking of the R/C members and should allow the structure to remain operational. Non-structural components may also have developed some minor cracking. However, no retrofit would be required with the exception of some patching (possibly epoxy injection which is described later) of the minor cracks in the structure.

(b) ***moderate damage - "repairable" condition***. The structure would be in need of repairs to regain a serviceable condition. The damage would be in the form of cracking in both the structural and non-structural elements. During the repair, the structure, or part thereof, may or may not be temporarily closed depending on the severity and location of the damage. The existing damaged structure must also be classified as either safe or unsafe from collapse in event of a future strong ground motion (possibly an after-shock), with the later meaning the temporary closure of the structure until a retrofit can be completed. Obviously, economics would play a vital role in the decision of whether to repair the existing damage or demolish the structure and possibly construct another. Nevertheless for a moderately damaged state, it is assumed that the retrofit of the structure is possible and more economically beneficial.

(c) ***severe damage - "irrepairable" condition***. The structure can be regarded as unsafe and in need of major restorations. Damage would result in the form of widespread cracking and spalling of the R/C structural and non-structural members. The onset of the resulting failure mechanism may be evident. The damage would initiate danger to the occupants of the structure and nearby individuals from the possibility of falling debris and the risk of collapse. Since the costs for repair could be considerable, the structure may be classified as irrepairable and thereby force immediate closure and demolition. However, it should be emphasized that such a damage state is expected for a very strong earthquake, when life safety is the greatest concern.

(d) ***partial or full collapse***. For completeness, the final damage state would be visually obvious and catastrophic. A separation area from the structure may be required for the safety of the local residents in case of falling debris until total demolition could be completed. This damage state may well cause loss of life and therefore should be avoided in new design and retrofit of existing structures.

The previous descriptions categorized the damage states of R/C structures subjected to strong ground motions as either minor, moderate, severe, or collapse. Herein the retrofit of structures excited by seismic loads with a moderate (repairable) assessment of structural damage will be focused, in particular related to the one-third scale three story R/C frame model described in the preceding sections.

In general after strong shaking, all structures should be thoroughly inspected by an engineer and if necessary analyzed for the capability of resisting future ground motions. Next if required and desired, several retrofit schemes should be considered and analyzed to repair the induced structural damage. Since the repair might be very costly, a rigorous retrofit design should be considered to improve the structural response for any future strong ground motions (although this may not be a design criterion in low seismicity zones). This design must comply to a target damage state. In low seismicity areas, the target damage state for design is within the irrepairable damage state (near collapse, but not collapse).

2.3 Local Member Damage versus Global Failure Mechanisms

Researchers and engineers have gained tremendous knowledge from past earthquakes by studying the local and global damage of various typical structural members and components of R/C buildings, especially in moderate seismicity zones where some major earthquakes caused widespread damage and collapse of non-seismically designed structures (Armenia 1990).

The following discussions are related to the expected damage in R/C structures, not designed to withstand seismic loadings (GLD structures). Some of the ***local member damage concentrations (or failures)*** that can develop in GLD R/C structures from strong ground motions are outlined below, along with their impact on the overall (global) structural response.

(a) Beams:

(i) *Flexural failure* from steel yielding and concrete crushing, which is desirable in a global failure mechanism.

(ii) *Shear failure* from beam hinging due to minimal transverse reinforcement. This corresponds to a loss of moment capacity in the beams which can lead to large floor displacements under seismic as well as service loading.

(b) Columns:

(i) *Flexural failure* from steel yielding and concrete crushing, which is undesirable in a global failure mechanism.

(ii) *Transverse steel (hoop) fracture or buckling of the longitudinal steel in the columns* may occur due to inadequate shear and/or confining steel. The inadequate shear and confining steel results in a lack of member ductility, which can result in the development of an undesirable local column failure (hinging).

(iii) *Lap splice failure* may occur from critical stress concentrations from lateral loads in the splice zone (above story slab). This leads to loss of moment resistance and thus may promote a soft-story mechanism.

(iv) *Cover spalling* which leads to compression failure and an undesirable column failure.

(c) Beam-Column Joints:

(i) *Pull-out of the discontinuous positive beam reinforcement in the beam-column joints* from the unexpected positive moments. This localized failure results in an overall structural stiffness degradation that leads to large story deformations in event of future ground motions.

(ii) *Joint shear failure* may occur due to lack of or inadequate joint shear reinforcement. The global consequences are similar to (i).

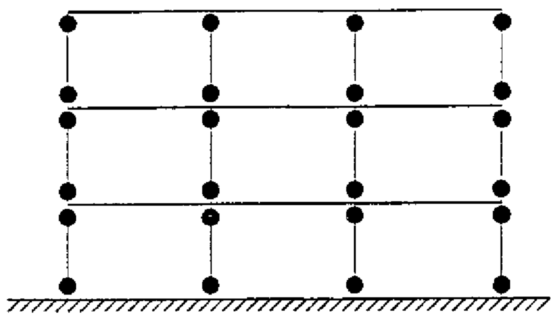
(iii) *Sliding bond failure* of the beam reinforcement in the joints from localized crushing of the concrete due to repeated inelastic cycling.

(iv) *Spalling of the concrete cover in the exterior beam column joints*. The spalling of the concrete cover can lead to a column failure due to depreciating axial load capacity. The cover spalling may be indicative to the lack of anchorage for bars within the joint, which result also in a structural stiffness degradation.

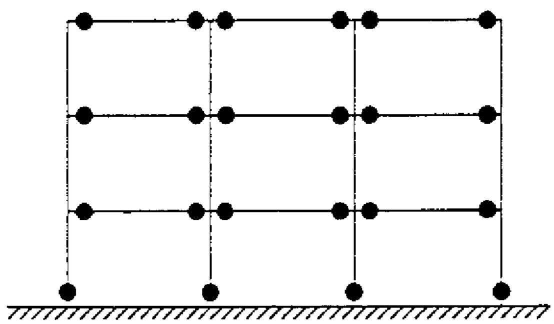
In addition to the various local damages, ***global structural failures or collapse mechanisms*** can develop. These are major causes for partial or total collapse of structures. The possible basic collapse mechanisms for R/C structures are shown in Fig. 2-1 and are outlined below:

(a) *Column-Sidesway/Soft-Story Collapse Mechanism*
(b) *Beam-Sidesway Collapse Mechanism*
(c) *Hybrid Collapse Mechanism* - combination of (a) and (b)

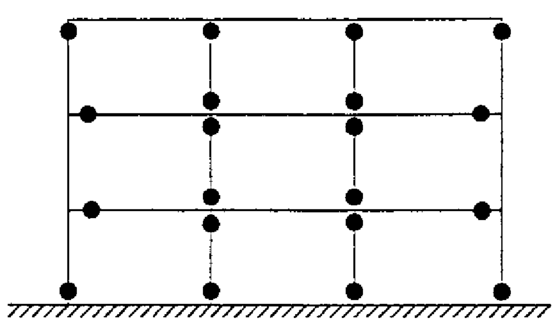
Note that although mixed (hybrid) mechanisms are a possibility, the discussion is continued only to the basic mechanisms listed above as (a) and (b). For a typical gravity load designed R/C framed building excited by strong ground motions, laterally induced shear forces develop in the columns from the inertial loads causing large bending moment demands on the columns. The gravity load design column moments are relatively small since the beam design moments on each side of the column face tend to cancel. The columns are essentially constructed with minimum size and reinforcement. Such non-seismic detailing practice results inherently in a weak column - strong beam construction. Due to low strength, lack of member ductility resulting from a lack of seismic detailing, and the high degree of seismically induced bending moment, local hinging can develop in the columns. Such local column hinging can lead to the undesirable development of a *structural column-sidesway/soft-story (Fig. 2-1a) collapse mechanism.* These global failure mechanisms are well documented from past earthquakes and result in a brittle (non-ductile) sudden collapse of structures. A beam-sidesway mechanism is ideally the preferred mechanism since energy is dissipated more efficiently by plastic hinges in the beams, Park and Paulay (1975), and the vertical loads can still be transfer through the undamaged columns.



(a) Column-Sidesway/Soft Story (apparent in previous testing)



(b) Beam-Sidesway (desirable)



(c) Possible Hybrid

FIG. 2-1 Collapse Mechanisms for the Model Structure

2.4 Concerns and Expected Seismic Damage of GLD R/C Frame Structures

Some of the concerns of gravity load designed R/C frame buildings during earthquakes are: (i) Insufficient strength to stay within a functional stage and avoid large inelastic deformations; (ii) Danger of severe loss of life due to non-structural damage, such as windows, blocks, ceiling tiles, etc., due to large deformations; and (iii) Small margin of safety against total structural collapse.

It was shown in a previous report (Bracci et al., 1992b) that the columns of the low-rise GLD R/C frame model building were heavily damaged, ranging from moderate to severe damage states, from the simulated shaking table ground motions. An incipient column-sidesway/soft-story collapse mechanism was evident in the response during the simulated motions. This type of damage is also expected to occur in prototype low-rise buildings that have similar structural details during severe seismic events. In high-rise buildings where the columns in the lower stories carry large amounts of gravity loads and are appropriately designed, a desirable strong column - weak beam behavior inherently exists. However the columns in the upper stories of high-rise buildings contain smaller amounts of gravity loads and can be vulnerable to weak column - strong beam behavior during seismic activity, as the behavior in low-rise buildings.

In zones of low to moderate seismicity, the probability of occurrence of a severe earthquake is very small. Due to the concerns and expected (or actual) seismic damage in GLD frame buildings, seismic retrofit should focus on strengthening the columns such that they are stronger than the beams to enforce a more desirable beam-sidesway mechanism (Fig. 2-1b) and avoid the more dangerous soft-story mechanism (Fig. 2-1a). This retrofit will force the local damage from the vulnerable columns to be distributed into a larger number of beam and slab components of the structure. Since the beam-sidesway mechanism consists of a large number of hinges in the beams and only a few at the base columns, the resulting mechanism has a larger safety margin against collapse due to the larger rotational capability and energy dissipation capacity of the beam hinges as compared to the columns in a soft-story mechanism. Column strengthening would also result in a stiffer structure which should imply better control of the story displacements under the influence of large lateral loads. However, column stiffening may have an adverse effect on the response, as additional accelerations may result in larger base shear demands. In such cases, larger shear and moment demands are imposed on individual components. Therefore the local retrofit should be carefully designed and balanced in the overall (global) structural context.

2.5 Local and Global Retrofit Methods for GLD R/C Structures

In Section 1, several local repair and retrofit (upgrade) techniques previously performed and tested were identified in a literature survey. In this section, several local retrofit methods are proposed along with a summary of the design process. These methods are analyzed and compared in the context with the three story model structure presented in this study. The integration of the local retrofit in a structural system is further discussed and the results of testing one of the solutions is presented in Section 4.

2.5.1 Improved Concrete Jacketing

An improved concrete jacketing method, shown in Fig. 2-2, is proposed to satisfy some of the deficiencies of columns integrated in a gravity load designed R/C frame building. This retrofit technique is outlined below:

1. *Encase existing columns in a concrete jacket with additional longitudinal and transverse reinforcement.* For the upper columns of the structure where an increased strength is the main retrofit objective, the increased column size and added reinforcement would be such that the retrofitted columns have greater moment capacities than the corresponding adjacent beam (overstrength) capacities. However at the base columns, the retrofit objective is not to increase moment strength but to increase the shear and ductility capacities, since the foundation is presumed to be relatively weak. Therefore the reinforcement is not anchored in the foundation to avoid transmission of any additional stresses to the foundation. Another constructive reason for discontinuing the added rebars is that plastic hinges should form at the base columns in the desirable beam-sidesway mechanism (see Fig. 2-1b). Therefore instead of strengthening these sections and possibly altering the desirable mechanism, a hinge can always form at the base by the discontinuation of these rebars. Proper confinement is necessary to provide rotational ductility to these hinges. To deter any shear failure in the base columns, the additional transverse reinforcement should also provide a dependable shear strength for the most adverse combination of column end moments.

2. *Post-tension the longitudinal high strength column reinforcement.* The required longitudinal reinforcement in the column is housed in a sleeve from the mid-height of the first story to the roof and unbonded to the concrete. Below the mid-height of the first story, the longitudinal reinforcement is bonded to the concrete for anchorage. The

longitudinal reinforcement is post-tensioned vertically. The bonded reinforcement from the foundation to the mid-height of the first story (or higher) is to provide the required anchorage reaction to the applied prestressing force. Post-tensioning the added high strength reinforcement has the following beneficial aspects on the composite section:

(a) Enhances the shear capacity of the column and beam-column joint zone from the increased axial load by ensuring the structural behavior is always in the elastic regime.

(b) Provides an initial strain in the new composite section of existing concrete and added grout to ensure compatibility of the section.

(c) Provides a compressive pressure on the discontinuous positive beam reinforcement to deter pull-out.

3. *Provide a reinforced concrete fillet in the unreinforced beam-column joints for: (a) enhanced joint shear capacity; and (b) anchorage for the discontinuous beam reinforcement.* In addition to providing increased joint shear capacity with the concrete fillet, the negative bending moment capacity of the beams at the column face would also be increased due to the added compressive width from the confined concrete in the web of the beam from the fillet. The weak link is therefore forced to the end of the fillet. Since the development length of the positive beam reinforcement is also increased, the full positive moment strength of the beam section would be able to develop without pull-out occurring at the column face. Therefore by ensuring strong column - weak beam behavior and providing a fillet, the critical beam hinge would be forced away from the column face to a point near the end of the fillet with moment strengths of the unretrofitted beam section. The dimensions of the fillet are designed from the required development length of the discontinuous beam reinforcement and the designated hinge locations in the beams.

The relocation of the potential beam plastic hinges from the face of the columns was studied by Paulay and Bull (1979) and by Park and Milburn (1983). It was suggested from their studies to move the potential beam hinge the smaller distance of either the beam height or 500 mm. from the column face. Buchanan (1979) reported on the construction of New Zealand's tallest concrete building which uses spandrel beams with the potential beam hinges moved toward the center of the span. Paulay and Priestley

(1992) also summarized some of this work. Al-Haddad and Wight (1986) analytically studied the effects of moving the plastic hinge locations in beams. According to their conclusions, it is suggested to locate the potential plastic hinges in the beams approximately one beam depth away from the column face. This enables the joint core to remain elastic and provides a longer anchorage length for the beam bars.

Application of the above mentioned retrofit procedures to the three story frame model is outlined below:

Choudhuri et al. (1992) (see Part I of the Retrofit Report Series) quasi-statically tested a retrofitted companion interior sub-assemblage component of the model (column-beam-slab) using the improved concrete jacket retrofit for the column to determine construction feasibility and capacity limits of components (see Section 1 for details). It was observed that the column stiffnesses, strengths, and ductilities were dramatically increased. Severe damage was transferred to the beams and slab with primarily elastic behavior in the columns. Thus the desirable beam-sidesway mechanism developed in the sub-assemblage under large lateral cyclic loads.

Since appropriate column strength, ductility, and a desirable failure mechanism had resulted in this component test using the concrete jacketing method, a similar retrofit scheme was adopted in this study. The scheme was selectively applied to columns of the model by increasing the existing 4 in. square section to a 6 in. square section using the same concrete as in the component test. Fig. 2-3 shows the details of the improved concrete jacketing technique in a typical retrofitted column of the model. The added reinforcement consists of high strength 3/8 in. diameter threadbars (f_y = 120 ksi) housed in a plastic sleeve above the bottom half of the first story and post-tensioned at the roof with a total force of about 31 kips (0.7 f_{pu}). The column section at the foundation has discontinuous added longitudinal reinforcement without prestressing and is considered as a regular reinforced concrete section.

From manufacturer (Master Builders, Inc.) specifications and tests conducted on the material during retrofit on 2 in. cubes, it is found that the concrete used in the jacket (Set-45) has characteristic properties of low shrinkage, high strength (28-day cylinder strength of about 8.0 ksi), modulus of elasticity of 5,250 ksi, and superior bond adhesion to the existing concrete columns. Since the special high strength concrete provides good bond to the existing concrete column, the retrofitted section can be idealized as a 6 in. square reinforcement concrete section with four layers of steel, two existing and two prestressed. For a conservative design, a homogeneous concrete strength of 5.0 ksi is used for idealizing the composite section. Since initial stresses also exist in the added threadbars from prestressing, the yield strengths are appropriately adjusted for tensile and compressive strength capacities with a corresponding prestressing force applied to the section. Under ultimate load, the strains in the post-tensioned threadbars are assumed to be proportional with the strain profile in the concrete. Fig. 2-4a shows the interaction diagrams for the section with an applied prestressing force in the added reinforcement to 70% of the ultimate strength (31 kips total) and with the same bars without prestressing. It should be noted that since the interaction development considers the initial strains in the concrete and steel from prestressing, the axial load in the interaction diagram refers to additional axial loads only. It can be observed that the tensile capacities with and without prestressing are identical. However the compressive capacity and moment capacity at the corresponding dead loads for the prestressed section are smaller due to the applied compressive forces from prestressing. Although the effective capacity is somewhat smaller in the prestressed columns, the additional column and joint shear capacity, the uniformity of strains, and improved bond of the rebars in the joint are important benefits of retrofit.

From the prestressed section in Fig. 2-4a, it can be observed that the moment capacity of the retrofitted section without any axial load is about 110 kip-in, which is precisely the moment capacity observed in the component test, Choudhuri et al. (1992). Considering prestressing with the additional axial force from the dead loads (total of about 45 kips), the moment capacity is determined to be about 130 kip-in (for a first story upper interior column). It was shown by Bracci et al. (1992a and 1992b) that a first story interior column had an over-strength capacity of about 44.0 kip-in. Therefore, the bending moment strength of retrofitted column is increased about 200% with the concrete jacketing method. The nominal strength of the retrofitted columns of a first story interior beam-column joint section is about 59% stronger than the corresponding beams considering slab steel contributions from the full slab width and no pull-out effects (99 kip-in and 65 kip-in for the negative and positive beam moments, respectively). Due to disproportionate distributions of moments during higher mode response of frame buildings, ACI-318 requires a 20% increase in factored design column strength as compared to the design strength capacity of the beams. This corresponds to nominal column strengths of about 71% stronger the beam capacities with a strength reduction factor $\phi_c = 0.7$ and 115% stronger if beam overstrength is considered. Therefore the column retrofit may not be adequate for a new design. For investigating the adequacy of a minimum retrofit, a lower bound retrofit solution was considered in this study appropriate for a low seismicity zone.

The interaction diagram of the base column, with discontinuous longitudinal reinforcement and a dead load of about 15 kips, is shown in Fig. 2-4b. It is developed based on a 6 in. square section with only the two existing layers of steel. It can be observed that a moment capacity of the base column is 70 kip-in. In comparison with the unretrofitted base column, the bending moment strength of this retrofitted column is increased about 59%.

Since the columns with the exception of the lower first story columns are retrofitted to remain primarily elastic, the non-seismically detailed beam-column joints must also remain elastic to avoid an undesirable joint shear failure. The existing interior columns of the model have no shear reinforcement in the joints. Therefore according to ACI-318 for an axial compression member, the code based shear capacity of the concrete, V_c, is defined as:

$$V_c = 2\left(1 + \frac{N_u}{2000A_g}\right)\sqrt{f_c'}\,b_w d \tag{2.1}$$

where N_u = Factored axial load normal to the cross section

f_c' = specified compressive strength of concrete

b_w = web width

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement

Therefore using Eq. (2.1), the unretrofitted joint shear capacity from the concrete according to ACI-318 is 2.6 kips. Since this is expected to be inadequate for retrofit, a concrete fillet is used with additional joint reinforcement for added shear strength and confinement of the joint. Also since the development length of the positive reinforcement in the beams is inadequate for developing the full moment capacity, the concrete fillet can be designed to provide the additional development length required for this reinforcement.

The design of the fillet stems from basic mechanics. Paulay (1989) showed that the required joint shear reinforcement is equal to the sum of the forces from the positive and negative reinforcing steel in the beams. Fig. 2-5 shows a free body diagram of a column with maximum stresses in the positive and negative beam reinforcement due to applied shear forces in the columns. From equilibrium, the shear forces in the column can be solved as follows:

$$V' = [(C_t + T_t) - (C_b + T_b)]\cdot\frac{1}{2} + [(C_t + T_t) + (C_b + T_b)]\;\frac{z_d}{2l_c} \tag{2.2a}$$

$$V = [(C_b + T_b) - (C_t + T_t)] \cdot \frac{1}{2} + [(C_t + T_t) + (C_b + T_b)] \cdot \frac{z_d}{2l_c} \qquad (2.2b)$$

where C_t, C_b = internal rebar compression forces, top and bottom, respectively

T_t, T_b = internal rebar tension forces, top and bottom, respectively

z_d = Distance between positive (bottom) and negative (top) beam reinforcement

l_c = Distance between story mid-heights

V' = Shear force at mid-height of the top column

V = Shear force at mid-height of the bottom column

Note that for symmetrically reinforced beams, V = V'.

The maximum shear force occurring in the joint, V_{joint}, can be described as follows:

$$V_{joint} = [(C_b + T_b) + (C_t + T_t)] \cdot \frac{1}{2} + [(C_t + T_t) + (C_b + T_b)] \cdot \frac{z_d}{2l_c} \qquad (2.3)$$

The dependable concrete shear strength, V_c, from ACI-318 for prestressed members is the smaller of V_{cw} and V_{ci} below:

$$V_{cw} = \left(3.5\sqrt{f_c'} + 0.3 f_{pc}\right) b_w d \qquad \text{for uncracked sections} \qquad (2-4a)$$

$$V_{ci} = 0.6\sqrt{f_c'}\, b_w d + \frac{V_i M_{cr}}{M_{max}} \qquad \text{for cracked sections} \qquad (2-4b)$$

but not less than:

$$V_c \geq 1.7\sqrt{f_c'}\, b_w d \qquad (2-4c)$$

where V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max}

M_{cr}, M_{max} = cracking moment and maximum factored moment at section

f_{pc} = compressive stress in the concrete due to prestressing and applied axial loads

The required joint steel shear capacity in the fillet, V_{sh}, can be represented in terms of the maximum joint shear force and the dependable concrete shear strength as follows:

$$V_{sh} = V_{joint} - V_c \tag{2.5}$$

The required joint steel area in the fillet, A_{sh}, can be determined as follows:

$$A_{sh} = \frac{V_{sh}}{f_{yh}} \tag{2.6}$$

where f_{yh} = transverse hoop yield strength

For the model structure, the internal beam compression and tension forces in the beam are:

$$T_t = 3A_sf_s = C_b \tag{2.7a}$$
$$T_b = 2A_sf_s = C_t \tag{2.7b}$$

where A_s = area of a D4 rebar (0.04 in.^{2})

f_s = beam steel stress at overstrength, taken as 1.25 f_y (1.25*68 ksi = 86 ksi)

C_b = force contribution from concrete and steel for equilibrium

Therefore inserting Eqs. (2.7a) and (2.7b) into Eqs. (2.2a) and (2.2b), the column shears can be represented as follows:

$$V' = A_sf_s + \frac{5A_sf_sz_d}{l_c} \tag{2.8a}$$

$$V = - A_sf_s + \frac{5A_sf_sz_d}{l_c} \tag{2.8b}$$

The maximum shear force which can occur in the beam-column joints of the model is described as:

$$V_{joint} = 5A_sf_s - \frac{5A_sf_sz_d}{l_c} \tag{2.9}$$

Therefore the maximum required joint shear capacity is about 15.2 kip (note the unretrofit joint shear capacity is 2.6 kips). Considering a total axial force of about 45 kips from dead and prestressing loads and b_w = 14 in., the dependable concrete shear capacity from Eq. (2.4) is 6.2 kips. Therefore using Eqs. (2.5) and (2.6), the required area of the added reinforcement (unannealed D4 hoops with A_s = 0.04 in.^{2} and f_y = 82 ksi) in the fillet is calculated as: A_{sh} = 0.11 in^2 (2.75 legs). Therefore the provided joint reinforcement used for retrofit of the model is two (4 legs) unannealed D4 rebars fully around the fillet (see Fig. 2-3). A similar interior joint reinforcement detail was recently presented by Paulay and Priestley (1992) for a well-detailed joint section.

For the transverse reinforcement for the retrofitted columns, the required spacing of the added transverse hoops can be determined from ACI-318 as follow:

$$s_{req} = \frac{2A_{sh}f_{yh}d}{V_s} \tag{2.10}$$

where d = distance from the outermost compression fiber to the center of the longitudinal reinforcement in the beam

A_{sh} = area of added hoop reinforcement

f_{yh} = yield strength of the added hoop reinforcement

V_s = required shear strength

In the analytical study in the next sub-section, it is shown that the base shear capacity for the model from a shake-down analysis is about 22 kips (about 100% larger than unretrofitted). Since the moment of inertia of the retrofitted column is about 4 times the unretrofitted columns, the retrofitted column shear is estimated as 1/5 of total base shear from the ratio of total column stiffnesses in the model. Therefore from Eq. (2.10) with a column shearing force of 4.4 kips (1/5 of total base shear) and using ga. 11 black hoop reinforcement (see Bracci et al., 1992a for properties), the required spacing for the shear reinforcement in the base columns with no prestressing is obtained as 1.6 in. The provided spacing is 1-1/2 in. (1.5 in.). In the prestressed section, no shear reinforcement is required.

Some comments can be made about the construction and aesthetic characteristics of the concrete jacketing techniques.

(a) Drilling holes through the slabs and beams are required for pouring the concrete into the columns and continuity of the longitudinal reinforcement. Construction process requires formwork and is relatively easy, although drilling through the beam reinforcement should be avoided if possible.

(b) Closure of the structure would only be located in areas of retrofit, provided the structure has enough reserve strength to resist a near future earthquake. Therefore each story of the structure, or part thereof, would temporarily be closed only when the retrofit of that story is being worked on.

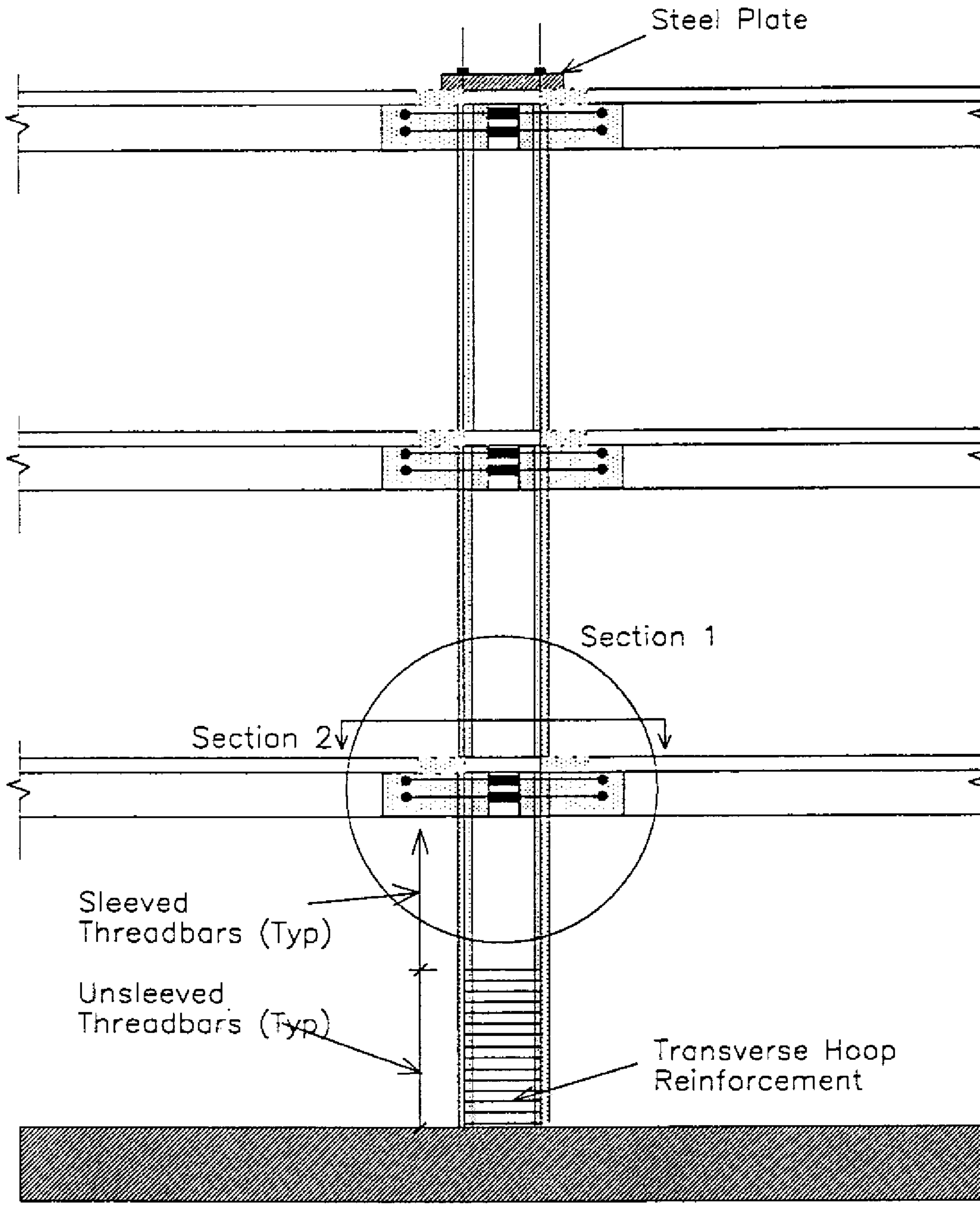
(c) Small reductions in the clear span widths would result for the retrofitted bay.

(d) Minimal amount of retrofit material, including transverse reinforcement, is required.

2.5.2 Masonry Block Jacketing

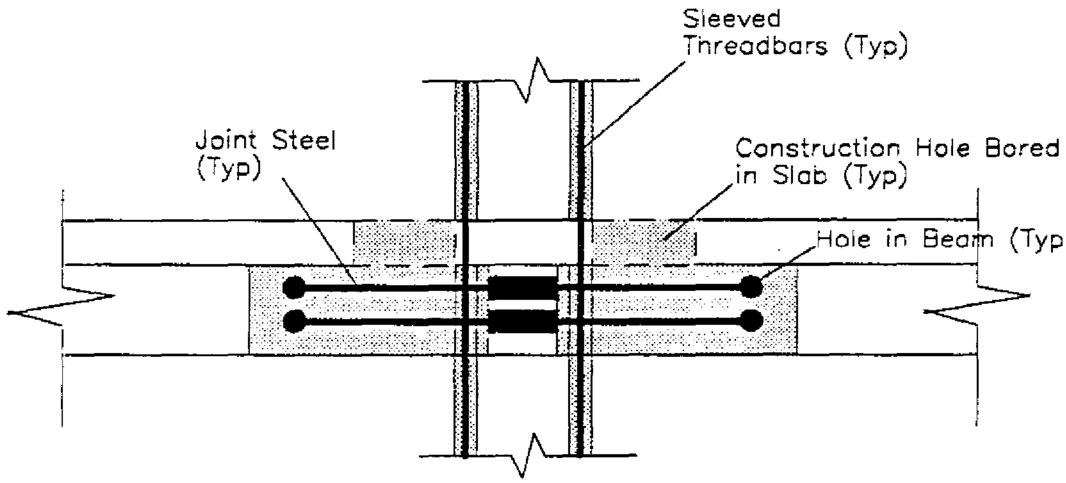
Similar to the concrete jacketing method, a masonry block jacketing method can be used for repairing and strengthening an existing damaged column. Fig. 2-6 shows a detail of a typical retrofitted column using the masonry jacketing technique. An existing damaged column can be strengthened by encompassing the existing section with masonry blocks. Some additional space between the existing concrete column and the new masonry blocks can be filled with grout and used for additional shear capacity and for addition of confining steel reinforcement to the existing column.

Additional longitudinal reinforcement, either prestressed or regular reinforcement, is provided in the jacketed zone extending continuously through the slabs. In this study, a prestressed reinforced concrete and masonry section is considered. The advantages of prestressing for this method are: (i) an increased shear capacity in the columns and joints; (ii) an initial uniform strain is obtained in the existing concrete and the new masonry blocks (for compatibility and for counteracting the stress losses from creep in the masonry joints); and (iii) a compressive pressure on the discontinuous positive beam reinforcement which would deter pull-out.

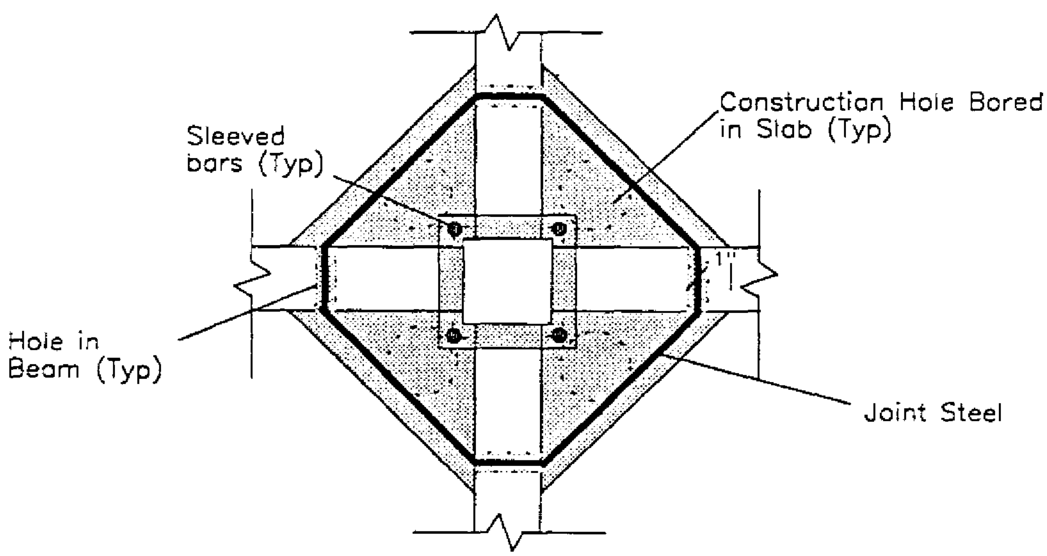


Elevation

FIG. 2-2a Improved Concrete Jacketing Technique

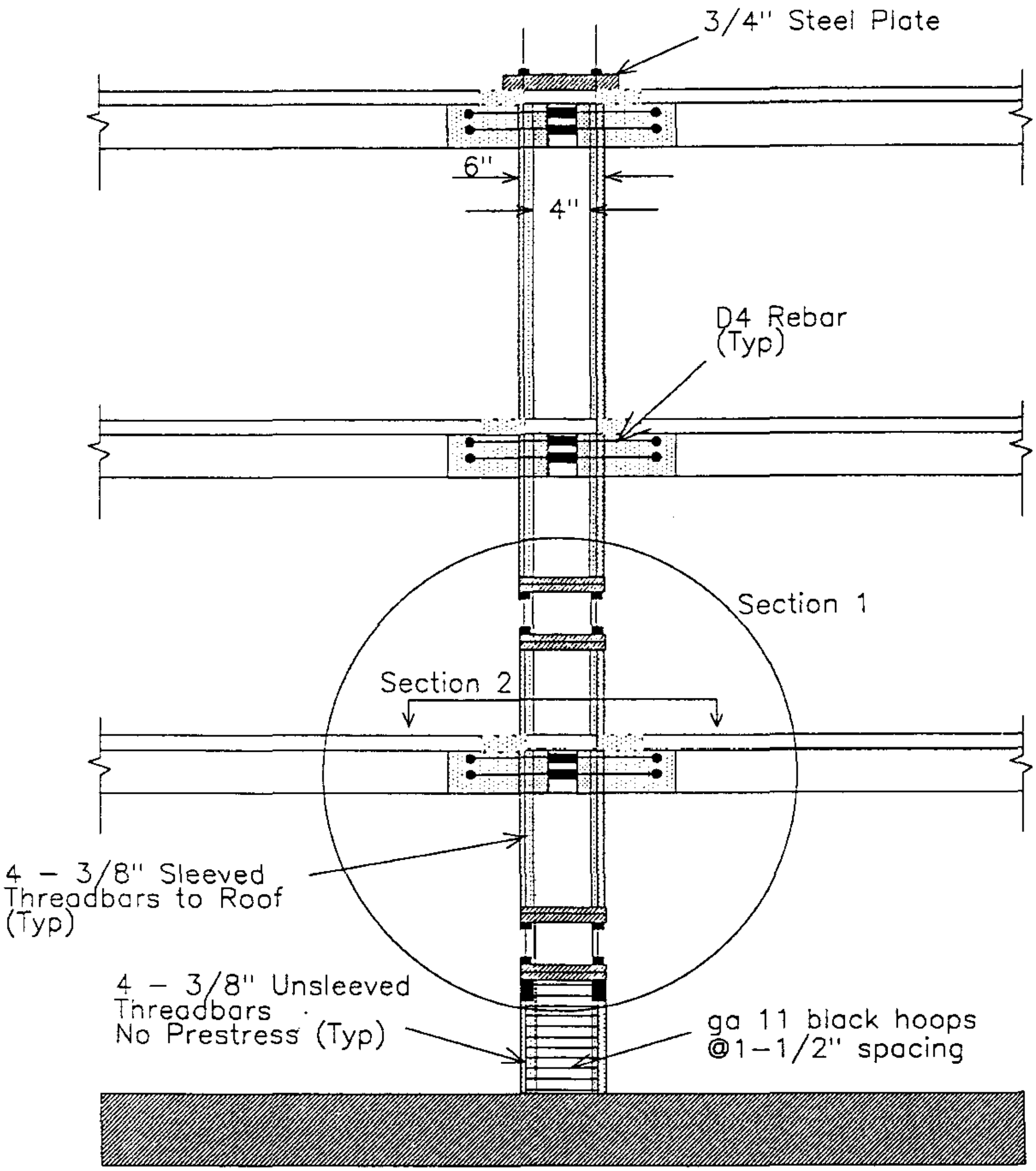


Section 1



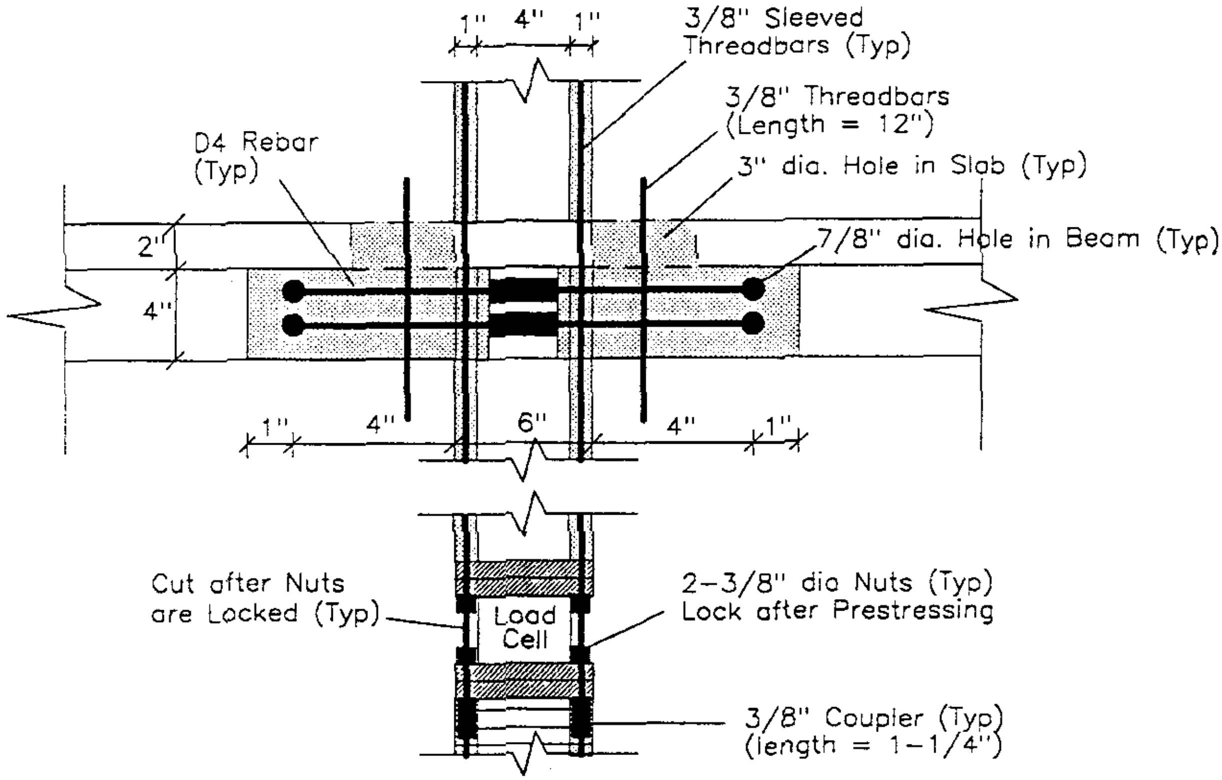
Section 2

FIG. 2-2b Improved Concrete Jacketing Technique (Cont'd)

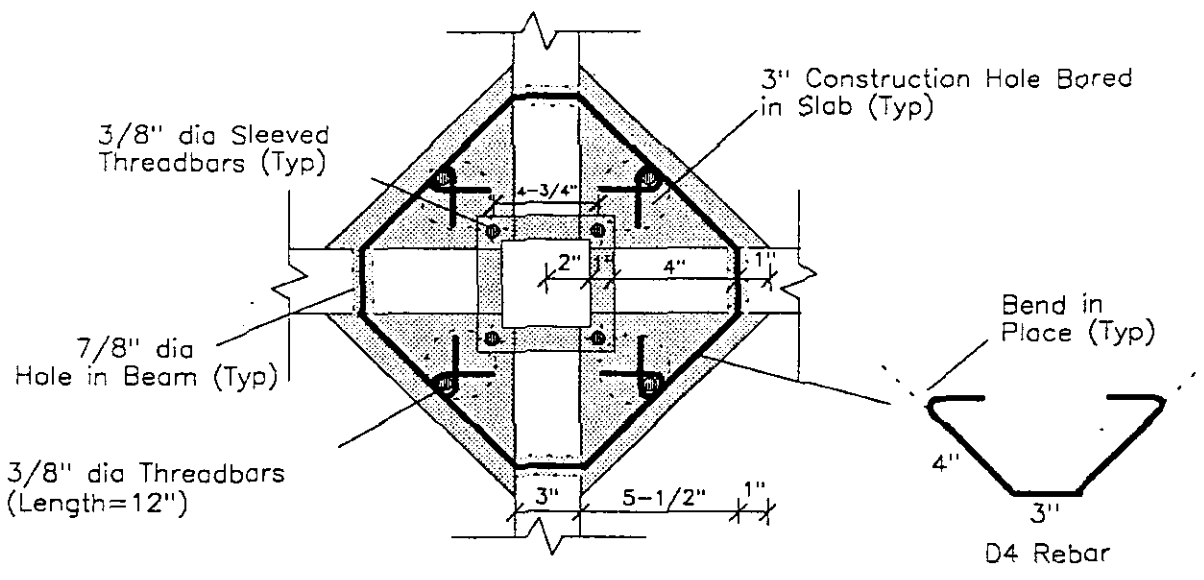


Elevation

FIG. 2-3a Improved Concrete Jacketing Technique for the Model

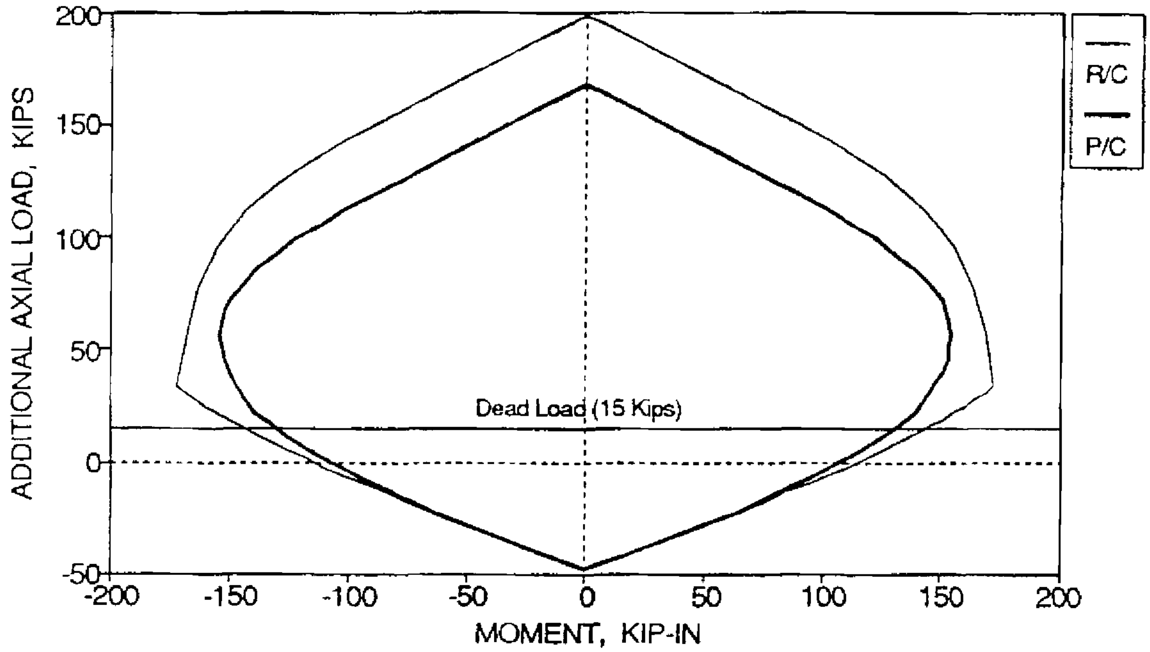


Section 1

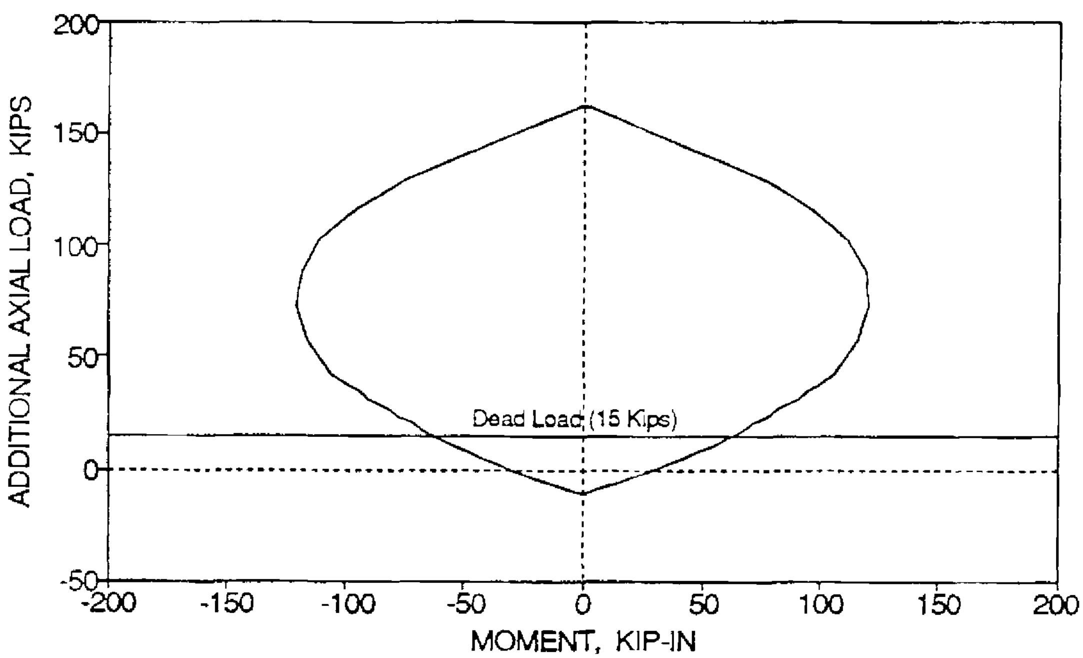


Section 2

FIG. 2-3b Improved Concrete Jacketing Technique for the Model (Cont'd)



(a) Prestressed/Non-Prestressed Section



(b) Base Section

FIG. 2-4 Interaction Diagram for the Columns using the Concrete Jacketing Technique

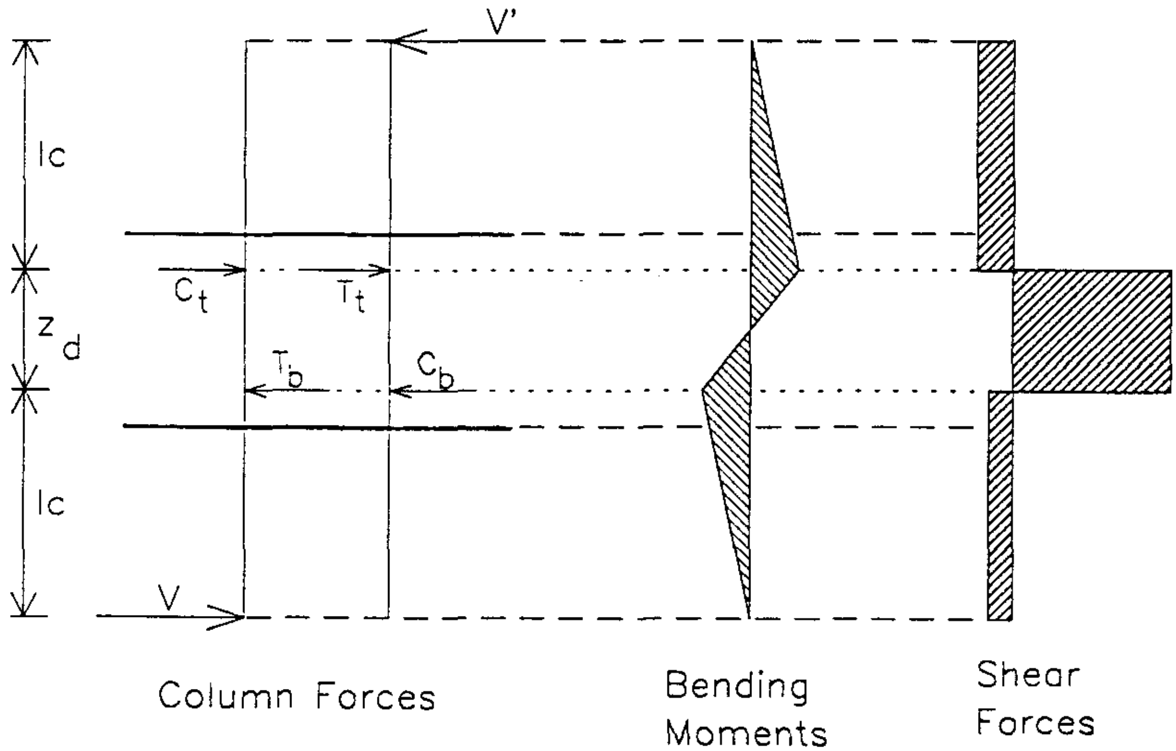


FIG. 2-5 Free-Body Diagram for a Joint with Maximum Shearing Forces