Since the lower first story columns are primarily located in the plastic hinge zone for a beam-sidesway mechanism (see Fig. 2-1b), transverse reinforcement is required in the section between the masonry blocks and the existing concrete columns for added shear strength and confinement as shown in Fig. 2-6. A thin wire mesh can be provided in the block layers to prevent shear cracking in the bed joints and ensure continuity of the masonry and existing concrete column section.

For the scaled model structure presented in this study, the existing 4 in. square damaged columns can be increased to a 9-1/3 in. square section by encompassing the column with the one-third scale of 6 in. masonry blocks. The strength of both materials, concrete and masonry, are considered in an interaction diagram ($f_m' = 1.2$ ksi, $f_c' = 4.0$ ksi) based on the compression depth at a particular load. It should be noted that the strength of the composite section will be governed by the quality of the work by the contractor. However, conservative estimates of material strengths are used for design. The suggested additional longitudinal reinforcement is 3/8 in. diameter threadbars with a yield strength of 120 ksi. Note that the added reinforcement and total prestressing force (32 kips) used for the masonry jacketing is identical with the one for the suggested concrete jacketing method. Accordingly, the tensile and compressive strengths of the reinforcement are appropriately adjusted. Under ultimate load, the strains in the prestressed reinforcement are assumed proportional with the strain profile in the concrete. Fig. 2-7a shows the interaction diagram for the masonry jacket column retrofit based on the composite section of masonry blocks and the existing reinforced concrete section with a prestressing force of 32 kips. With the prescribed axial dead loads of 15 kips for a first floor interior column, the predicted moment strength of the column with a masonry jacket retrofit is about 160 kip-in, which is about a 250% increase in strength from the original column. The retrofitted column strengths at a first story interior beam-column joint are about 95% stronger than the beams nominal capacity, which is more than required by the ACI-318 for design (71%). At the base column with discontinuous longitudinal reinforcement and without prestressing, the moment capacity can be observed as 80.0 kip-in from Fig. 2-7b, which is about an 80% increase in strength from the original column.

For the beam-column joints, the added thin wire mesh and transverse reinforcement in the block joints may be designed to adequately resist the shear forces in the joint from seismic loads. If these shear forces can not be resisted, a concrete fillet with joint reinforcement can be used for additional shear strength and confinement of the joint, similar to the one in the concrete jacketing.

Since the positive and negative beam reinforcement remain the same, the joint steel design would be identical to the improved concrete jacketing method. Thus the same concrete fillet design for the beam-column joints can be used for the different retrofit methods.

Some construction and aesthetic characteristics of the masonry jacketing method can be mentioned:

(a) The construction requires drilling of holes through the existing slabs and beams for pouring the concrete into the columns and fillet (if needed) and for continuity of longitudinal reinforcement. The construction process is relatively easy.

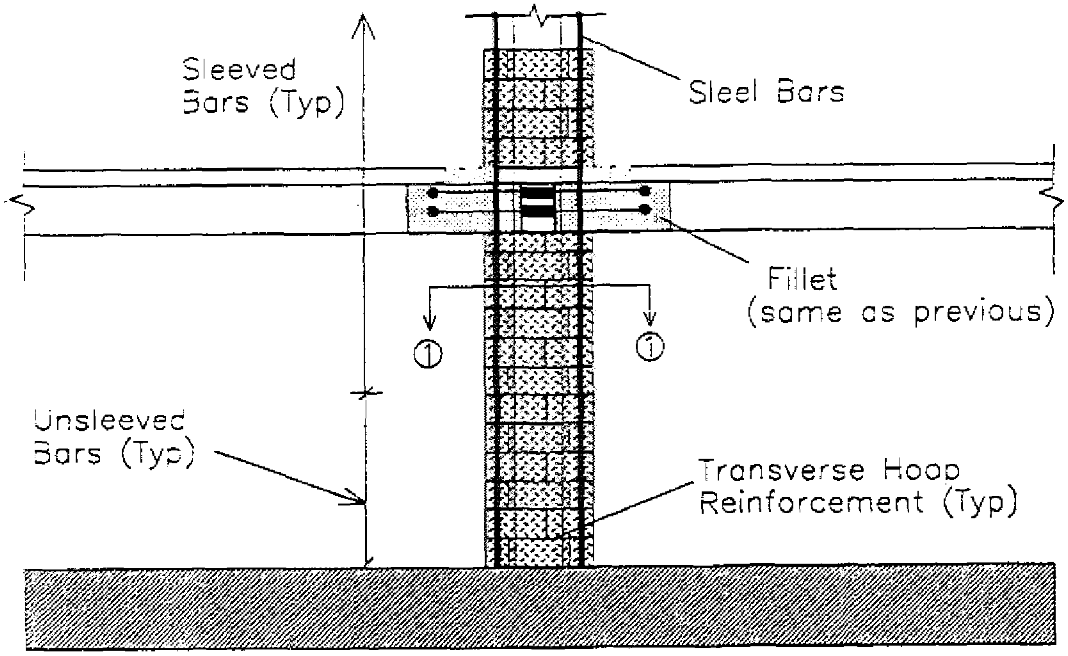
(b) Since the masonry block themselves are used as formwork for the grout, no additional formwork, except for the fillet (if needed), is required for the retrofit.

(c) Access in the structure would be limited only in areas of retrofit. Therefore each story of the structure should be temporarily closed when retrofit of that story columns are being worked on;

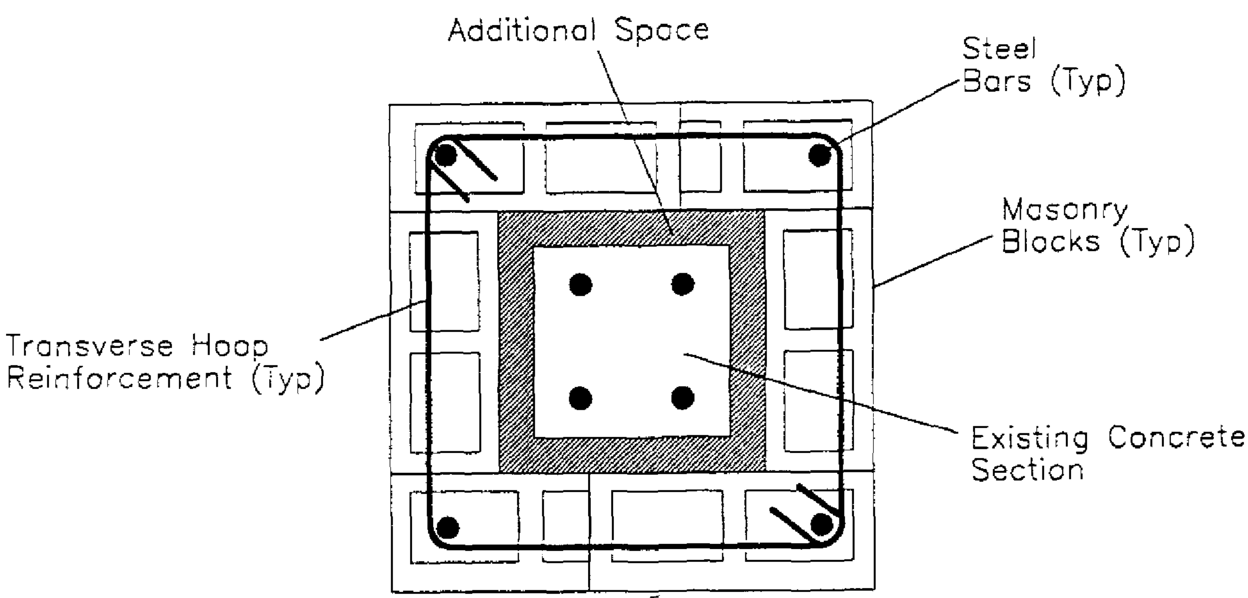
(d) The method leads to small reductions of the clear span (5-1/3 in.) for the retrofitted bay of the model. This span reduction, quantified above for the model, corresponds to a 16 in. (1'-4") span reduction of a 18 ft. bay in the prototype building.

2.5.3 Partial Masonry Infill

Masonry and R/C infill walls have been widely tested at universities/research institutions and constructed in practice for increasing the stiffness and strength of structures to control story displacements from high wind loads and other natural forces, including seismic loads. Some of these investigators include: Benjamin and Williams (1958); Stafford Smith and Carter (1969); Esteva (1966); Fiorato, Sozen, and Gamble (1970); Klingner and Bertero (1976); Kahn and Hanson (1976); Parducci and Mezzi (1980); Priestly (1980); Bertero and Brokken (1983); Krause and Wight (1990); and many more. Most new low to medium rise construction of R/C buildings in the Eastern and Central United States have such walls. Infill walls can also be constructed to retrofit an existing damaged (or undamaged) R/C building for improved structural stiffness and strength, thereby reducing story displacements. High shear is placed on the columns and beam-column joints potentially leading to a premature snap-through failure in an existing column. An architectural disadvantage of using an infill wall retrofit for an existing building is the loss of space and access in the building near the wall.

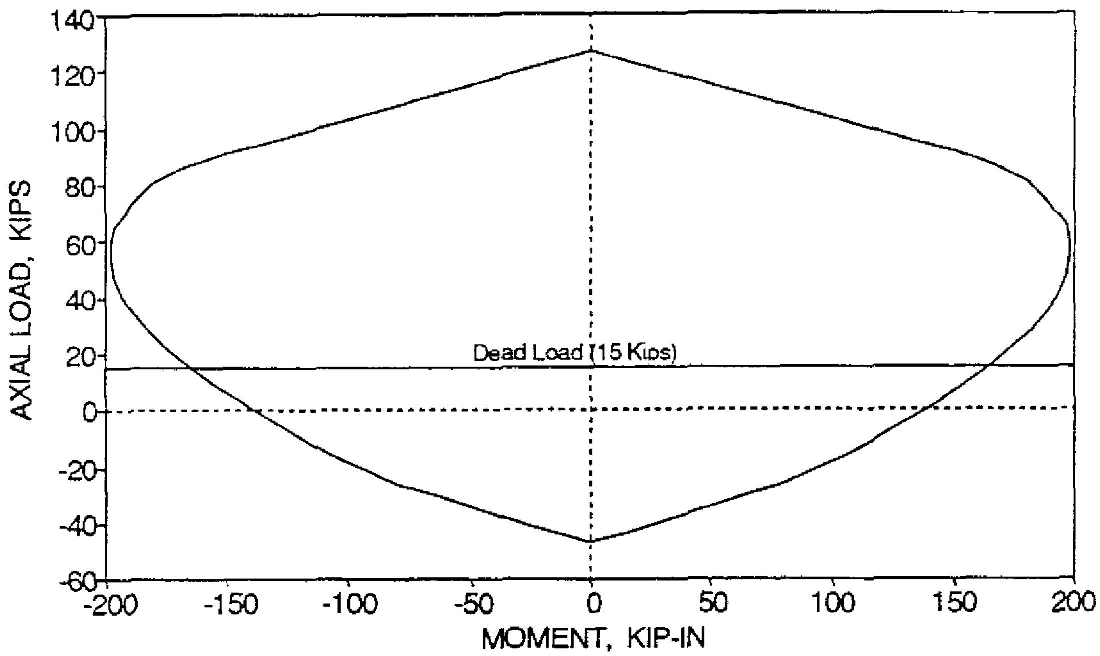


Elevation

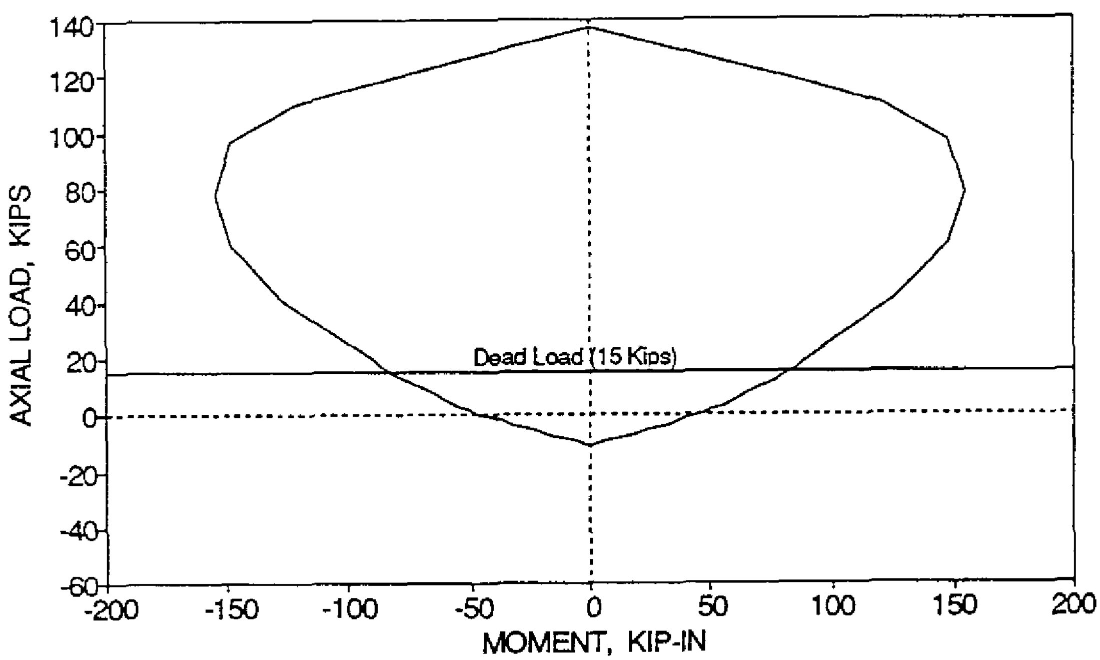


Section 1-1

FIG. 2-6 Masonry Jacketing Technique



(a) Prestressed Section



(b) Base Section

FIG. 2-7 Interaction Diagram for the Columns using the Masonry Jacketing Technique

To maintain an almost full passageway between bays and still enhance the critical column strengths, another method of retrofit can be suggested for low to moderate seismicity zones. A partial masonry infill wall can be constructed on each side of selected columns of the structure. Fig. 2-8 shows a detail using partial masonry infill walls on each side of the column for retrofit. It can be observed that the partial wall should extend not more than a few blocks from the existing column face. The number of blocks required in each partial infill wall may vary based on the desired column strength, but will be generally governed by the required development length of the discontinuous positive beam reinforcement. Longitudinal reinforcement in the masonry walls extends continuously through the story slabs for continuity of the wall. In this solution, post-tensioning can also be used to provide the benefits discussed earlier. Since the lower first story columns are primarily located in the plastic hinge zone for a beam-sidesway mechanism (see Fig. 2-1b), adequate transverse reinforcement should be provided in the masonry joints of this zone to resist the large shear forces from the seismic loads. Continuous transverse reinforcement should also be provided in the section between the masonry blocks around the existing concrete columns for added shear strength and confinement, as shown in Fig. 2-8.

For the scale model considered in this study, the existing 4 in. square damaged columns are strengthened using one-third scale 8 in. masonry blocks. The additional reinforcement is 3/8 in. diameter threadbars with a yield strength of 120 ksi. Note that the total prestressing force used for the partial infill method is identical as in the other methods. However since the reinforcement provided is twice the other methods, the prestressing force per bar is only half of the other methods. Fig. 2-9a shows the interaction diagram for the partial masonry infill retrofit based on the composite section of one-third scale 8 in. masonry blocks and existing R/C column with a total prestressed force of 32 kips. Therefore with the prescribed axial dead loads, the partial infill retrofitted columns for the first story of the model have a bending moment strength of 450 kip-in, which is about 10 times stronger than of the existing column. The column strengths at an interior beam-column joint are about five times stronger than the beams, which is well in excess of that required by the ACI-318. It is important to note that the moment strengths of the retrofitted columns assumed adequate transverse reinforcement so that the full moment strength of the section could be achieved. Also note that the column sections are intentionally designed to have a high moment capacity to force a beam-sidesway mechanism. At the base column with discontinuous longitudinal reinforcement and without prestressing, the moment capacity can be observed as 190.0 kip-in from Fig. 2-9b, which is about 330% stronger than of the existing column. However this may be excessive for the foundation.

To guard against beam-column joint failure, holes can be drilled through the beams and additional transverse reinforcement can be designed for the joint to adequately resists the shear forces from seismic loads. The masonry blocks can be cut in place to encompass the joint.

Some construction and aesthetic characteristics of the partial masonry infill method can be mentioned:

(a) The construction require holes cut through the slab for pouring of concrete grout and continuity of reinforcement. Construction process is very simple and economically beneficial.

(b) No formwork is required for the retrofit, since the masonry block themselves can be used as formwork for the grout.

(c) Access in the structure would be limited only in areas of retrofit. Therefore each story of the structure would temporarily be closed when retrofit of that story columns are being worked on;

(d) A clear span reduction of 16 in. (1'-4") for the retrofitted model bay. This span reductions corresponds to a 48 in. (4 ft.) span reduction of a 18 ft. bay in the prototype building.

2.5.4 Summary of Design Process

This sub-section summarizes the aforementioned design methodologies developed in Part I (Choudhuri et al., 1992) of the Retrofit Report Series. Note that in each scenario there is a parallel set of steps that progress through the design process.

CONVENTIONAL CAPACITY DESIGN	CAPACITY ANALYSIS AND REDESIGN

Step 1: Longitudinal Beam Reinforcement

D.1 Flexural design of beams:

Beams are designed and proportioned for moments which are a result of applying the moment redistribution process to the elastic design code actions. Beam plastic hinges are

R.1 Flexural check of beam strength distribution:

The anchorage of the positive reinforcement at the beam-column joint connections should be particularly considered. If the bottom bars are

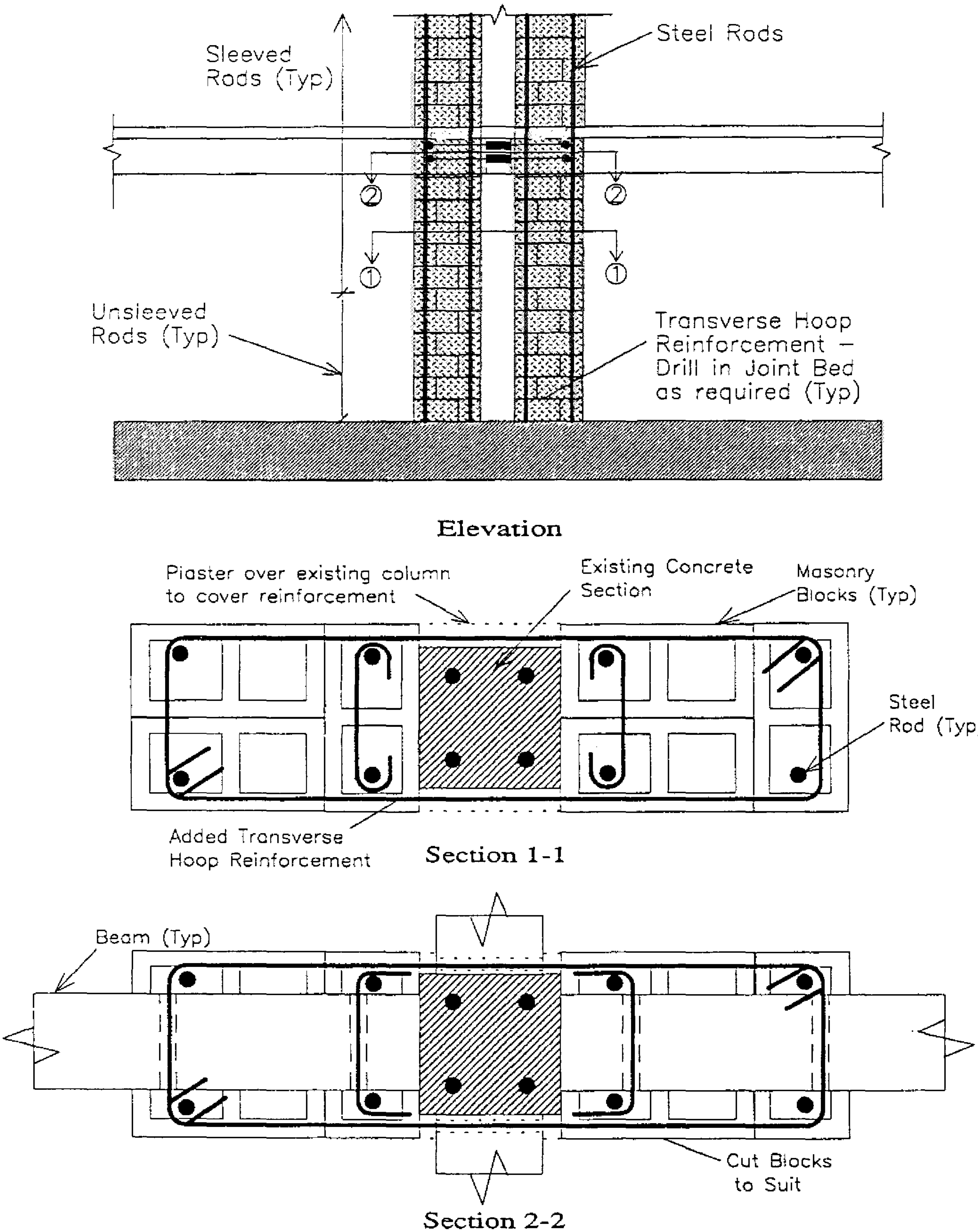
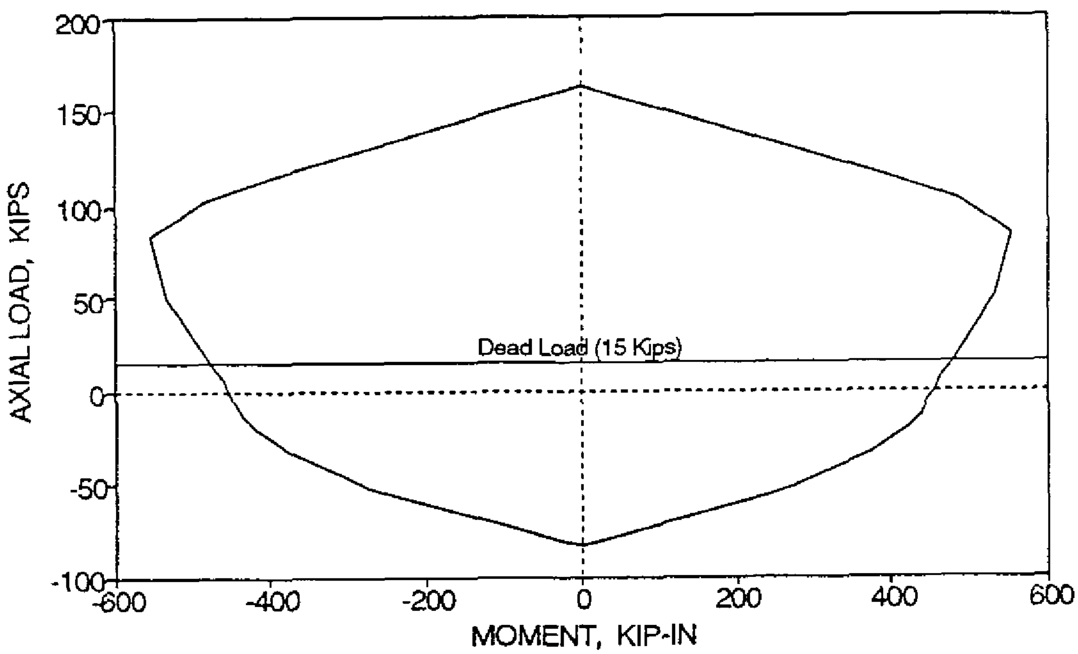
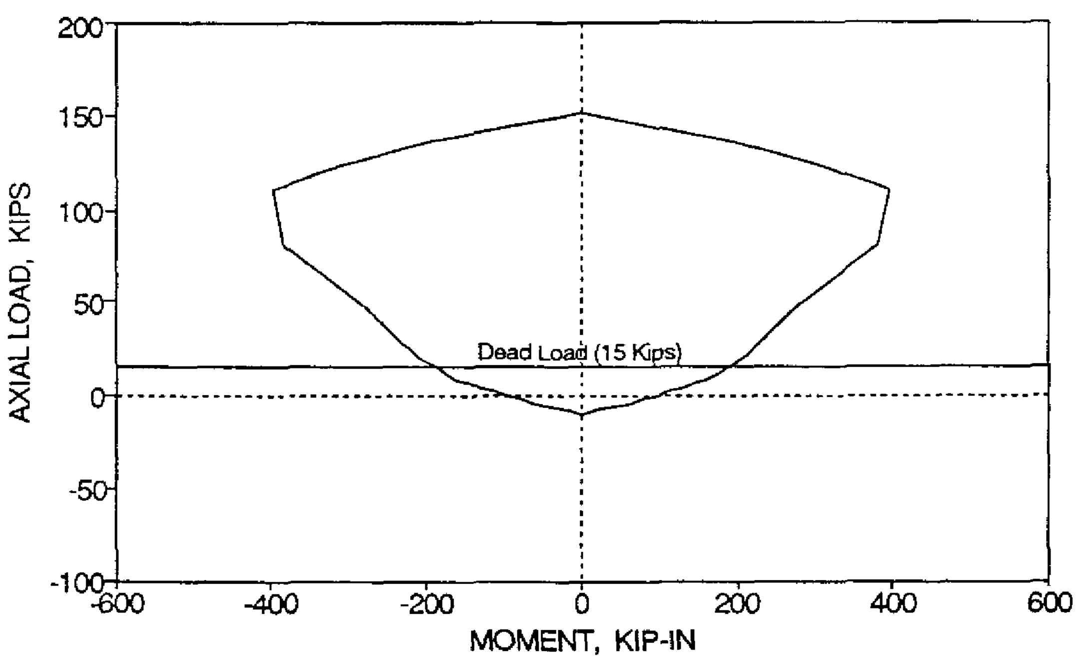


FIG. 2-8 Partial Masonry Infill Technique



(a) Prestressed Section



(b) Base Section

FIG. 2-9 Interaction Diagram for the Columns using the Partial Masonry Infill Technique *generally located at the column face and adequately detailed for ductility. From the actual reinforcement provided, the beam flexural overstrength capacity is assessed. This is used in beam shear and column strength design.*

discontinuous, then a means of providing dependable positive moment capacity needs to be devised for enhancing seismic resistance. A beam-column joint fillet is a recommended solution. From the actual reinforcement provided, the beam flexural overstrength capacity is assessed. This should include the full effects of the floor slab steel on the negative moment capacity.

Step 2: Transverse Beam Reinforcement

D.2 Shear design of beams:

This is achieved by providing shear strength for the entire beam to be greater than the shear corresponding to the maximum possible flexural strength at the plastic hinge region of the beam. The underlying premise being that inelastic shear deformations do not provide the essential characteristics for energy dissipation.

R.2 Check of shear strength:

It may not be the intent to bring beam shear capacity up to code strength for new design. However, critical regions such as potential plastic hinge zones and the centers of beams which may have little or no shear reinforcement should be assessed for shear strength and supplementary stirrups provided, if necessary.

Step 3: Longitudinal Column Reinforcement

D.3 Flexural strength design of columns:

The nominal flexural strength of the columns is computed by considering the beam overstrengths. This ensures a weak beam - strong column failure mechanism. It should be mentioned that the beam flexural overstrengths are determined and then an additional allowance is made to account for possible higher-mode structural response. From the actual longitudinal reinforcement

R.3 Flexural strength redesign of columns:

The required flexural strength of the columns is computed from the assessed beam overstrengths. The optimum axial load ratio is computed, which helps size the column section. The lower story column is designed as a conventional R/C section. If the imposed axial load due to gravity for the upper story columns is less than the optimum amount, prestressing can be applied to the *provided, the flexural capacity is assessed. This is used in the next step for column shear design.*

upper story columns. The cracking surface for the prestressed columns is plotted on a column interaction diagram and the reserve capacity is computed. The ultimate shear to be resisted by the columns should be calculated for the transverse reinforcement design.

Step 4: Transverse Column Reinforcement

D.4 Transverse reinforcement detailing for the columns:

From the most adverse combination of column end overstrength moments, the maximum possible shear force in the columns is computed. Transverse shear reinforcement is provided over the entire column height Additional shear steel and/or confinement or antibuckling steel is generally required in the potential plastic hinge zone.

R.4 Transverse reinforcement detailing for the redesigned columns:

For the lower story R/C columns, step D.4 applies for the shear steel design. For the upper story PSC columns, use the prestressed concrete code equations to determine the shear resisted by the concrete. Generally the intrinsic shear strength of the compressed concrete would be greater than the ultimate shear to be resisted, else provide supplementary transverse shear steel.

Step 5: Beam-Column Joint Reinforcement

D.5 Detailing of the beam-column joint:

The beam-column joint is a poor source of energy dissipation and thus needs to be detailed to resist the high shear input from the beam and column actions. In this step, the designer should attempt to keep the joint elastic by reducing, if not eliminating, any inelastic deformation due to the joint shear forces and bond deterioration.

R.5 Detailing of the beam-column joint:

Check that there is adequate longitudinal beam bar anchorage through the joint core. Since the length of the joint fillet has been decided in step 1 of the redesign, the designer should attempt to detail the fillet reinforcement in this step. The joint may be considered to behave in an elastic manner and the shear resisted by the concrete in elastic joints is computed. If the input shear forces from the beams and columns exceed that resisted by the concrete via strut action, provide the necessary reinforcement.

2.6 Global Retrofit of R/C Structures - Analytical Evaluation

The retrofit solutions outlined in the previous sub-sections provide local retrofit measures to columns, joints, beams, and components. However, the effectiveness of integrating these local retrofit schemes in a structure is not entirely obvious. Application of certain retrofit measures may not be beneficial to the overall performance of the structure. Therefore a global verification of integrating the local retrofit schemes is performed analytically using IDARC, Kunnath et al. (1990), with structural parameters described from engineering approximations to obtain an assessment of the effectiveness of integrating "the parts into the whole".

The objective of the analytical study is to first evaluate the seismic response of the existing model to another strong ground motion. If the seismic performance is not acceptable, evaluate the seismic response of the retrofitted model with the proposed concrete jacketing, masonry jacketing, and partial masonry infill alternatives. The control parameters for selecting the optimal global retrofit scheme for the model under strong ground motion are: story displacements (inter-story drifts); base shear demands and capacities; stress demands in members; and the apparent global collapse mechanism.

2.6.1 Analytical Evaluation of Original (Damaged) Model

It was shown by Bracci et al. (1992b) in Part III of the Evaluation Retrofit Series that a considerable amount of inelastic deformation and damage formed in the model during the moderate and severe ground motions. It was also shown that the analytical modeling using IDARC adequately predicts the response characteristics obtained in the experimental tests performed on the model. IDARC, therefore, is used again as an analytical tool to evaluate the strength of the model to resist another strong ground motion with further member stiffness deterioration. Consecutive runs of the moderate (0.20 g), severe (0.30 g), and another future severe (0.30 g) earthquakes are used to capture the hysteretic degradation in the model.

Table 2-1 shows the initial first period of the model, the base shear demands and capacities (from a shakedown analysis at 2% structure drift limit), the inter-story drifts, and the bending moment demands and capacities for the beams and columns obtained in the analytical study. The nominal column to beam strength ratios are also tabulated in Table 2-1. Fig. 2-10 outlines the calculation of the nominal column to beam strength ratios for an interior and exterior subassemblage and for a story subframe. The experimental structural response from the previous shaking table test (TFT_30) are presented along with: (i) the analytically calculated structural response for this test; and (ii) the analytically predicted structural response for a future occurrence of severe ground motion. For comparative evaluations, the Taft N21E component, scaled for a PGA of 0.30 g, is used to simulate the future severe demands. Note that this magnitude may be excessive for low seismicity zones. However it is considered that a more straight forward evaluation can be made for an extreme event and then compared with the observed performance. It can be observed that good agreement exists between the experimental response and the analytical response for TFT_30 (also see Bracci et al., 1992b). From the future severe ground motion analysis, increases in inter-story drifts are observed, which can be attributed to the softening of the model. The base shear demand is greater than the analytical base shear capacity from a shakedown analysis based on a 2% drift limit. This will almost ensure severe damage or collapse in a future shaking. The bending moments are also increasing. Note that the nominal column to beam strength ratios for an interior subassemblage and a story are 0.60 and 0.75, respectively (weak column - strong beam behavior). With increasing story displacements, base shears, and moments in the members up to full capacities for a future severe ground motion, further damage would be expected in the model with strong probability of collapse occurring. If constructed in an area of high seismicity, this structure would be rated unserviceable and subject to closing.

2.6.2 Analytical Evaluation with Proposed Retrofit Methods

Since the model was assessed as a moderately damaged structure (Bracci et al., 1992b), repair and retrofit is required before serviceability could be reinstated. An analytical study of the suggested local retrofit methods, presented in the previous sub-section, integrated in the model are presented using IDARC with the Taft N21E PGA 0.30 g ground motions. Since many members make up the structural system of the model, a few options of retrofitting the members of the structure existed. With induced seismic excitation, the interior columns would be more critical than the exterior since larger demand bending moments, shear forces, and axial loads will develop (Bracci et al., 1992b). El-Attar et al. (1991b) observed failure in the first story interior columns of the 1/8 scale model replica under a very large base motion. Therefore it was determined to evaluate the global response for retrofitting: (i) only the interior columns;

and (ii) all the columns for each bay for both the concrete and masonry jacketing methods. Since a partial masonry infill wall could not extend beyond the existing exterior facade of the structure, the stiffening of only the interior columns for each bay is examined.

In addition to the various retrofit techniques for columns, (iii) continuous (full base fixity) and (iv) discontinuous (partial base fixity) reinforcement is considered in the critical lower first story columns for the concrete jacketing alternative. Note that full fixity may create foundation problems. Nevertheless, a special connection to the foundation would be required to obtain the increased bending moment capacity.

The initial column stiffnesses used in the analyses are different for the various retrofit techniques and are chosen as 1.0 EI_g and 0.7 EI_g, respectively for the concrete jacketing and masonry retrofit methods. It was shown by Bracci et al. (1992a) that the initial column stiffnesses used in STAAD to match the first period of the R/C model were 0.565 $(EI_{col})_g$. Also to fit the experimental response, the initial column stiffnesses were about 0.60 $(EI_{col})_g$. However since post-tensioning is applied in the proposed retrofit alternatives, the equivalent member stiffnesses are expected to be in the range from 0.8 to 1.0 $(EI_{col})_g$ from the higher axial loads. However since the concrete used in the jacket has superior bond adhesion to the existing column, the full EI_g of the section is used. For the masonry blocks and grout, the bond to the existing column is not as superior and some cracking may still result. Thus 70% of $(EI_{col})_g$ is considered appropriate. These initial member stiffnesses are assumed to be uniform throughout the height of the structure. At the lower first story columns with the partial base fixity (discontinuous rebars and no prestressing), the respective equivalent stiffnesses used are 0.5 EI_g, 0.5 EI_g, and 0.33 EI_g, respectively for the concrete jacketing, masonry jacketing, and partial infills methods. These lower values reflect the more cracked nature of these reinforced sections. Paulay and Priestley (1992) suggest ranges for an effective moment of inertia between 0.7 I_g and 0.9 I_g for heavily loaded columns and between 0.5 I_g and 0.7 I_g for columns with axial loads of about 0.2 $f_c' A_g$. Therefore comparable initial column stiffnesses are approximated for the retrofitted columns.

Since the beams developed only minor damage from the previous shaking, the initial stiffnesses of the beams are about 0.45 $(EI_{bm})_g$ in the analytical study. Note that this beam stiffness is similar to the engineering approximations used for the undamaged building by Bracci et al (1992b). Since the exterior columns were moderately damaged from the previous shaking, the initial stiffnesses used in the unretrofitted exterior columns are about 0.27 $(EI_{col})_g$. Note that this is a reduction of about 30% from the initial properties used in the experimental fit from the previous shaking.

For development of the hysteretic rule, a post-cracking stiffness of EI/2 is assumed for all retrofit methods. The yield strengths of the beams and columns are computed from basic mechanics principles. Note that the beam moments consider slab steel contributions from the full slab width. Also note that the exterior beam yielding moment in the positive direction considers the effect of slip of the discontinuous bottom beam reinforcement (50% reduction in rebar area based on the prototype ratio of provided and required embedment lengths). However with retrofit, the interior beam moments consider full moment capacity without the pull-out effect. The hysteretic properties for the beams and columns in the analytical modeling for all the retrofit methods are defined based on previous component testing as: (i) 0.3 and 0.8 for the stiffness degradation factor for the columns and beams, respectively; (ii) 0.1 for the strength degradation factor; (iii) 1.0 for the target slip factor; (iv) 1.00 for the slip reduction factor; (v) 1.5% and 1.0% for the post-yielding stiffness ratio for the columns and beams, respectively; and (vi) 2% for the damping ratio.

The platform program IDARC, Kunnath et al. (1990), was used to carry out the inelastic analysis for a severe earthquake (Taft N21E 0.30 g) based on member behavior developed from engineering approximations. The global and local response results for the different retrofit methods are summarized in Table 2-1. The initial first mode periods vary between 0.25 sec. and 0.36 sec. It can be observed from the spectrum in Fig. 2-11 that this period range is in the vicinity of major amplifications from the Taft N21E ground motions (response spectrum of an elastic single degree-of-freedom system for 2% and 5% damping). Although the acceleration amplifications are increased from the added stiffness, a beam-sidesway mechanism, stipulated in the retrofit design, will transfer damage from the columns to the more ductile beams. This can be observed in the redistribution of moment demands versus capacities in the beams and columns.

The resulting demands in the beams for all the retrofit methods are well beyond yield, but not beyond ultimate capacity. Note the large beam moment demands with the retrofit methods as compared to the analytical beam moment demands of the unretrofitted building. Also note that due to the reinforced fillet and added pressure from prestressing, the positive moment capacity of the beams is stronger and corresponding demands are greater, since the pull-out effect is eliminated.

For the columns, the extent of yielding varies depending on the global retrofit scheme applied. For the schemes using the weak base retrofit (weak link to foundation), some moment demands are slightly above the nominal ultimate capacity (incipient yielding). However, the demands are well within the dynamic ultimate capacity. For the case of strong base retrofit, large moment demands in the lower first story columns are observed, which are well beyond the nominal ultimate capacity. In a prototype structure, these large moment demands would need to be reacted by the existing foundation, if it is strong enough. Otherwise the existing foundation would need to be strengthened.

For the retrofit of the interior columns only, the moments developed in the exterior columns are well below their ultimate capacity, but some incipient yielding occurs in the third story columns. However for the retrofit schemes considering stiffening of all columns, the resulting moment demands in the columns are below yield.

The nominal column to beam strength ratio for the retrofit methods vary between 1.59 and 5.85 for an interior subassemblage. The story strength ratios vary between 1.49 and 4.69. Therefore strong column - weak beam behavior is enforced by the design.

Fig. 2-12 shows the resulting analytical damage states in the model for the different retrofit methods after a severe earthquake (Taft N21E with a PGA of 0.3 g). The resulting failure mechanisms for strengthening all the columns in the model are in the form of the classical beam-sidesway collapse mechanism. Strengthening only the interior columns results in a beam-sidesway mechanism but with added incipient yielding in some interior (retrofitted) and exterior columns. However the resulting moments in these members are well below ultimate capacity and the damage from cracking might be ignored.

It can also be observed from Table 2-1 that all the retrofit methods analyzed provide adequate control of the inter-story drifts, with the largest inter-story drift being less than the recommended by NEHRP (1991). The concrete jacketing of all the columns with the weak base criteria provides the best control of the inter-story drifts for the base motions. The base shear demands are also less than the ultimate capacities determined from a shake-down analysis in all cases.

Note that the greater margin between the base shear demands and capacities for the weak base retrofit as compared to the strong base retrofit. Without retrofit, the base shear demand is either equal to or greater than the capacities.

2.7 Summary Discussions

Three global/local alternatives were suggested for retrofit of R/C frame structures: (i) improved concrete jacketing; (ii) masonry jacketing; and (iii) partial masonry infill. These three techniques were evaluated analytically in global context on the damaged three story R/C model for: (a) retrofitting only the interior columns; (b) retrofitting all columns; (c) partial base fixity (discontinuous added reinforcement at foundation); and (d) full base fixity (continuous added reinforcement into foundation). To ensure an elastic beam-column joint behavior, a reinforced concrete fillet was also provided in the retrofitted joints. Post-tensioning of the added longitudinal reinforcement was proposed to increase the shear strength and thus avoid additional transverse reinforcement to improve the constructability of the retrofit. The post-tensioning also provides an initial uniform strain on the composite section and a compressive pressure on the discontinuous positive beam reinforcement to deter pull-out.

From the analytical evaluation, it was found that:

(1) Stiffening of the structure causes a shift in natural frequency which is in the vicinity of major acceleration amplifications for the Taft N21E accelerogram. An increased base shear demand develops.

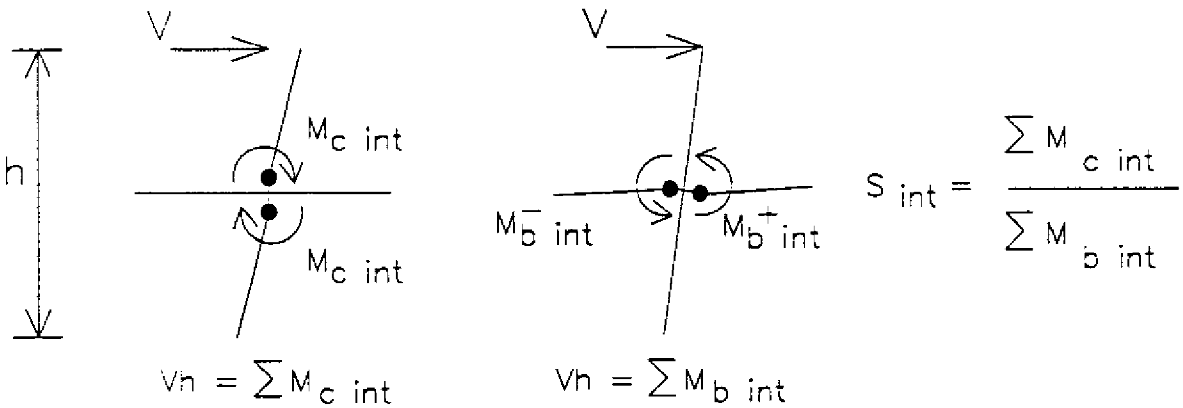
(2) A beam-sidesway mechanism after retrofit replaces the column-sidesway collapse mechanism as obtained in the original structure. However some combination mechanisms and incipient member yielding can also be observed from the resulting damage states.

(3) Moment demands in the beams are well beyond yield, but not beyond capacity. An increased positive moment capacity is achieved with the concrete fillet and partly by prestressing, which deters pull-out.

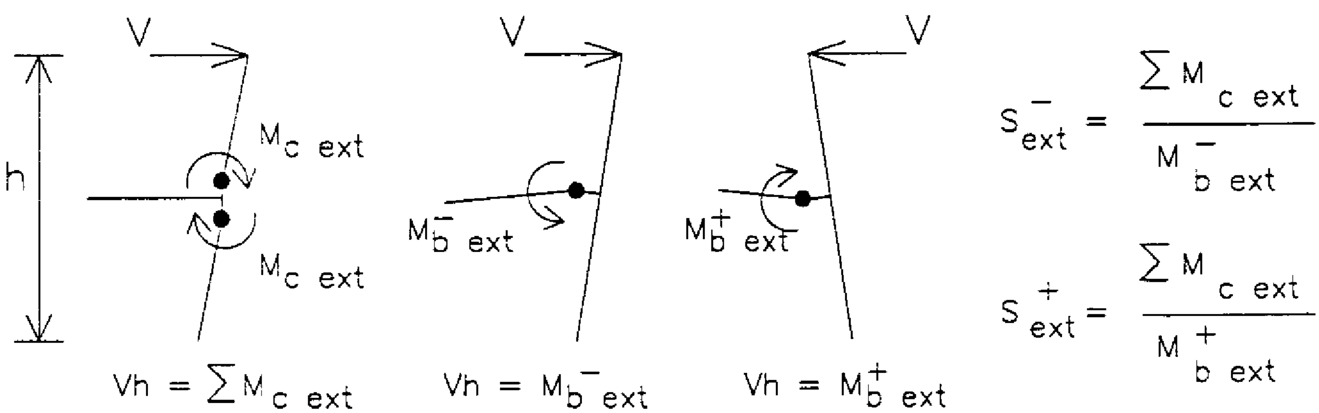
(4) Some incipient yielding in the columns occur for the weak base retrofit. For the full base fixity retrofit, large yielding moments develop in the base columns. These large moments can create foundation problems.

(5) Adequate control according to NEHRP (1991) of the inter-story drifts is obtained by the various methods.

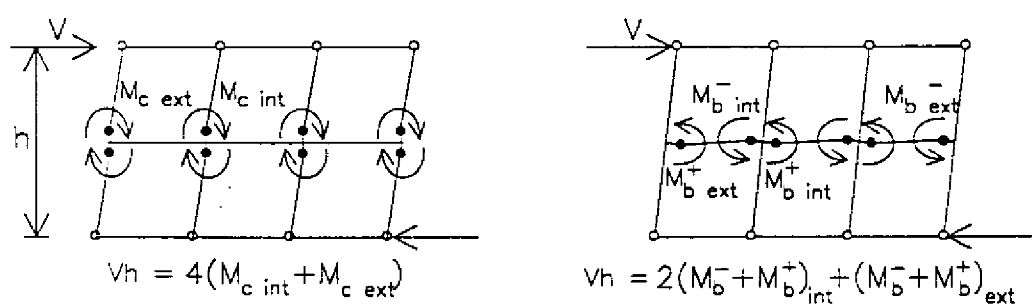
(6) The base shear demands are less than the ultimate capacities determined from a psuedo-static shakedown analysis based on a 2% drift limit and the margin between demand and capacity is slightly expanded.



(a) Interior Beam-Column Subassemblage



(b) Exterior Beam-Column Subassemblage



$$S_{story} = \frac{4(M_{c\ int} + M_{c\ ext})}{2(M_b^- + M_b^+)_{int} + (M_b^- + M_b^+)_{ext}}$$

(c) Story Subframe

FIG. 2-10 Nominal Column to Beam Strength Ratio Calculations

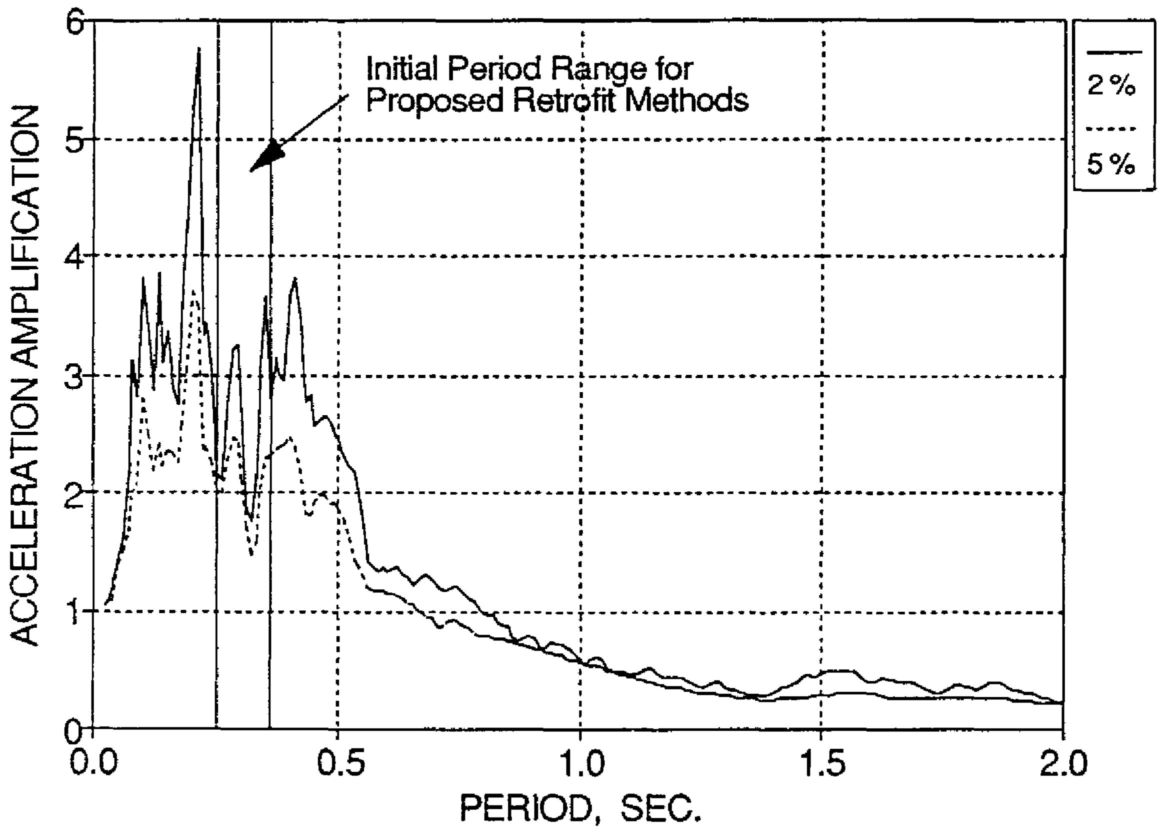


FIG. 2-11 Initial Periods of Retrofitted Buildings - Taft N21E Elastic Response Spectra

TABLE 2-1 Analytical Evaluation of Retrofit Techniques for Taft N21E (PGA 0 3 g)

	Classification	Type	Period (sec)	Maximum Global Response					Interior (kip-in)		Exterior (kip-in)		Column to Beam Strength Ratio		
				Base Shear (% W)	Drift (%)			3rd St. Disp(in)	Column Moments	Beam Moments	Column Moments	Beam Moments	Interior Comp	Exterior Comp.	Story
					1st St.	2nd St	3rd St								
EXISTING STRUCTURE	1. Model (Experiment-TFT_30)	Demand Capacity	0.70	15.3	2 03	2.24	0.89	2.35	44.2 38 0	84.6/+13.1 -98 8/+27 6	39 1 30.0	-39 4/+36 7 -85.3/+24 9	0 60	-0.71/ +2 40	0 75
	2 Model (Analytical-TFT_30)	Demand Capacity	0.70	14 8 15 0	1 81	2 42	0.65	2 28	39 3 38.0	-48 9/+30 2 -98.8/+27.6	30 1 30.0	-55 5/+35 2 -85 3/+24.9	0 60	-0.71/ +2 40	0.75
	3 Model (Future Taft 0 3g)	Demand Capacity	0.83	14 7 15 0	1 82	2 69	0.55	2 39	39 2 38 0	-41.8/+31 0 -98 8/+27 6	31 6 30 0	-42.6/+34.4 -85 3/+24 9	0 60	-0 71/ +2.40	0 75
RETROFIT WITH CONCRETE JACKETING	4. Interior Strengthened, Full Base Fixity	Demand Capacity	0.33	26 9 30.0	1.06	1.00	0.58	1.16	152 3 130.0	-76 2/+47 2 -98.8/+64 9	20 6 30 0	-49 1/+28 2 -85 3/+24 9	1.59	-0.71/ +2.40	1.49
	5 Interior Strengthened, Partial Base Fixity	Demand Capacity	0.36	24 4 28 5	1 24	1 10	0 40	1.30	132.5 130.0	-81 5/+49 7 -98 8/+64 9	20 7 30 0	-50.1/+28.8 -85.3/+24 9	1.59	-0.71/ +2.40	1.49
	6. All Strengthened, Full Base Fixity	Demand Capacity	0.27	36 2 39 0	0.79	1 17	0.89	1 33	135 1 130.0	-78.0/+50.1 -98.8/+64.9	134.8 130.0	-56 1/+37 2 -85.3/+59 7	1.59	-3.06/ +4.33	2.20
	7. All Strengthened, Partial Base Fixity	Demand Capacity	0.30	30 4 34.5	0.91	1.00	0.69	1.21	121.9 130.0	-76 1/+49 9 -98.8/+64.9	85 0 130.0	-54 8/+34 0 -85.3/+59 7	1.59	-3 06/ +4.33	2 20
RETROFIT WITH MASONRY JACKETING	8. Interior Strengthened, Partial Base Fixity	Demand Capacity	0.34	27 5 30.0	1.33	1 36	0 74	1 62	163 0 160.0	-83 3/+51.0 -98.8/+64.9	21.6 30.0	-50.5/+29 4 -85 3/+24.9	1 95	-0.71/ +2.40	1 76
	9. All Strengthened Partial Base Fixity	Demand Capacity	0.28	33.2 36 0	1.20	1.16	0.76	1.49	131 6 160 0	-79 9/+51 6 -98 8/+64 9	97.9 160 0	-55.9/+36 0 -85 3/+59.7	1 95	-3 76/ +5 33	2 71
RETROFIT WITH PARTIAL INFILLS	10. Interior Strengthened, Partial Base Fixity	Demand Capacity	0 25	35.9 37.5	0.78	1 03	0.96	1.33	209 3 480.0	-78 2/+47.2 98 8/+64 9	22 1 30 0	-50 2/+28 2 85 3/+24.9	5.85	-0 71/ +2 40	4.69

Note: Base shear capacities determined from a shake-down analysis
Classifications 1 and 2 compare the experimental and analytical results for TFT_30
Column to beam strength ratios based on Nominal Moment Capacities

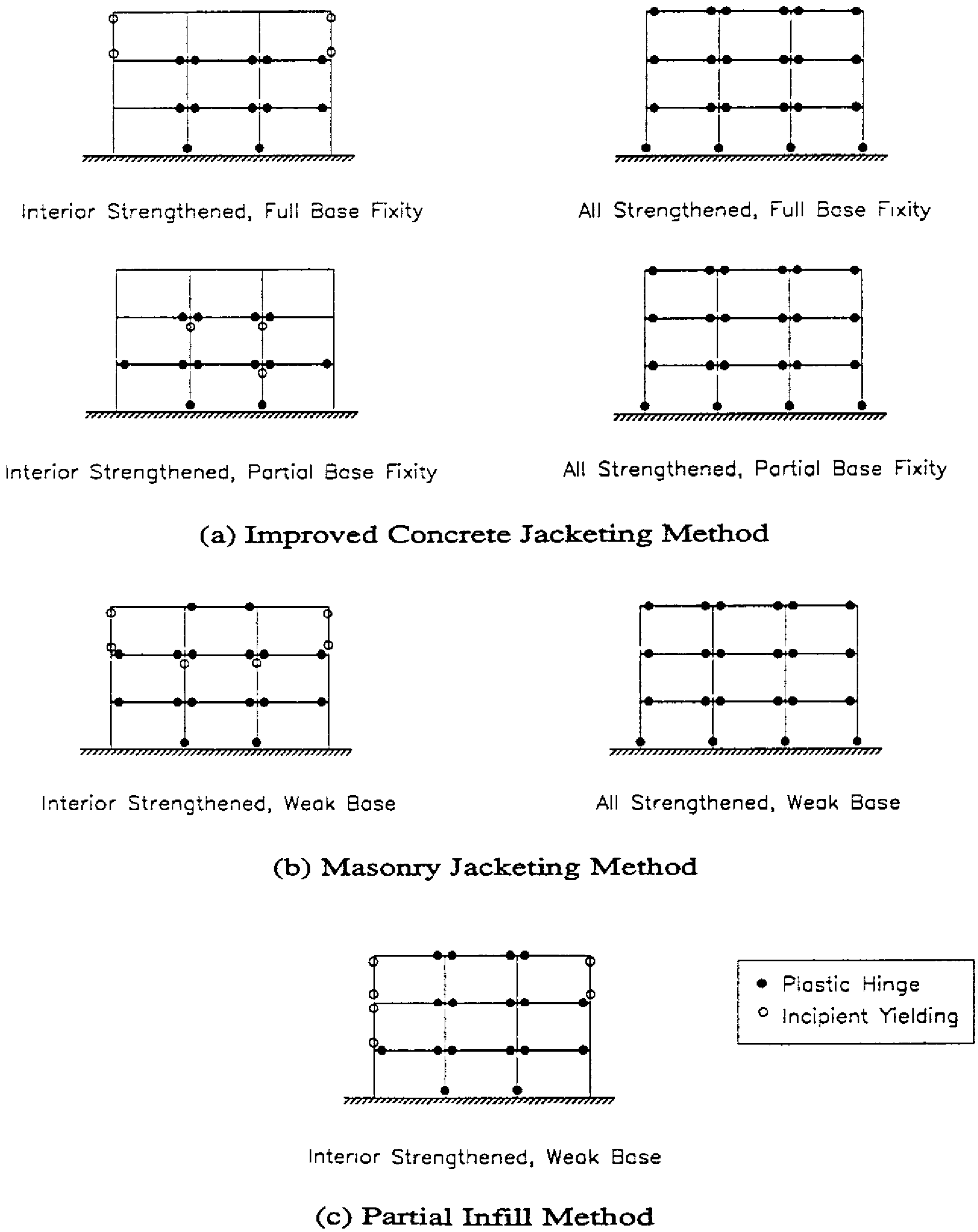


FIG. 2-12 Damage States for the Retrofitted Model (Analytical)