

Mathematical Modeling of Infilled Frames

By Richard E. Klingner

Synopsis: Two approaches are discussed for mathematical modeling of the elastic and inelastic response of infilled frames. The first approach is based on idealizations of local behavior, while the second is based on observations of overall behavior. Both approaches are found to give good representations of nonlinear response. The second approach, based on the equivalent strut concept, is believed to be efficient for use in analyzing the response of complex, infilled frame structures.

Keywords: computer programs; cracking (fracturing); dynamic loads; earthquake resistant structures; elastic analysis; frames; infilled frames; lateral pressure; mathematical models; reinforced concrete; structural analysis.

ACI member Richard E. Klingner is an Assistant Professor of Civil Engineering at The University of Texas at Austin and Secretary of ACI Committee 442, Lateral Forces. His Ph.D. research involved experimental and analytical investigations of the earthquake resistance of infilled frames.

BACKGROUND

There is increasing recognition among structural engineers that elements which have traditionally been considered nonstructural can significantly affect the behavior of a structure subjected to dynamic loads such as wind and earthquake. This recognition is due, among other things, to the development of improved analytical techniques for modeling the mechanical behavior of non-structural elements in the inelastic as well as elastic ranges. These techniques may be classified into two broad groups: (1) local or microscopic approaches, in which idealized local behavior (cracking, yielding, bond-slip, etc.) is used to predict force-deflection relationships at the subassembly level, and (2) macroscopic approaches, in which idealizations are developed predicting observed subassembly behavior at an overall level only. This paper will discuss examples of each approach which are relevant to the modeling of infilled frames. These approaches are general. However, the specific examples discussed herein apply to reinforced masonry infills bounded by reinforced concrete frames.

Many investigators have studied the effects of infill panels on the response of frame structures to lateral loads. Experimental studies (5,7,8,10,15) have shown that in the elastic range infill panels stiffen and strengthen the frame. In the inelastic range, infill panels can contribute significantly to the frame's energy dissipation capacity through the development of friction across distributed panel cracks (4,8).

Under very low levels of lateral load, the infill panel does not crack, nor does it separate from the bounding frame. Lateral force-deflection behavior is linear and elastic. Small lateral loads cause the formation of cracks along the boundary between the frame and the panel. Although these boundary cracks do not significantly reduce the lateral stiffness of the infilled frame (8), their presence can be included in conventional finite element analyses using bond-link elements (9,13).

Increased lateral loads, however, damage the panels themselves. Figures 1-3, taken from Ref. 8, show characteristic steps in the deterioration of an infilled frame subjected to increasing levels of reversed lateral load. Figures 1 and 2 show the formation of extensive cracks parallel to the compression diagonal, followed by crushing and spalling of the infill along the compression diagonal. As shown in Fig. 3, deterioration of the

infill panel at one story level causes a reduction in the infilled frame's lateral story stiffness at that level. Further cycles of reversed loading cause severe deterioration in that panel, and lead to the formation of a sidesway mechanism in the frame (5,8).

It is evident that infilled frame structures exhibit many kinds of nonlinear inelastic behavior following the onset of extensive panel cracking. The panel cracks cause a reduction in the panel's shear stiffness. Increasing stress levels lead to nonlinear stress-strain behavior in local regions of the panel. After portions of the panel spall or crush, panel cracks increase greatly in width, and significant slip displacements occur along these cracks. Reversed cycling to large lateral displacements results in severe deterioration along the cracks, and causes the formation of hinge regions in the bounding frame. Lateral load resistance of the infilled frame at this stage depends on the stiffness of the frame's beam-column connections, the available rotation capacity of the hinge regions, and the rate at which the panels deteriorate.

After the formation of extensive panel cracks, infilled frame behavior cannot be modeled accurately using linear elastic idealizations. The resistance of the cracked shear panel must be idealized using either local or macroscopic approaches. An example of each type of approach will be discussed, and the results will be compared.

ANALYSES USING LOCAL MODELS

Infill panels and other types of shear elements have been analyzed by several researchers using local models (2,3,6). An example of this type of analysis, applied to reinforced concrete shear panels, is discussed by Darwin and Pecknold in Ref. 3. Plain concrete was modeled as a nonlinear orthotropic material. The effects of softening under compressive stresses were included, as well as the formation of cracks under tensile stresses. Crack closing under load reversal was accounted for. Figure 4, taken from Ref. 3, shows that Darwin and Pecknold's proposed model agrees satisfactorily with the experimental load-deflection curve of a cyclically loaded shear panel (2).

Local models for infilled frames can consider other phenomena: the nonlinear moment-curvature behavior of the frame members can be modeled, as discussed by Aktan, Pecknold, and Sozen (1); local bond slip in the panels and frame members can be modeled using idealizations such as that of Morita (12); boundary cracks between the frame and panel can be idealized using techniques similar to those of Refs. 3 and 6. As noted previously, local models can give good reproductions of overall load-deflection characteristics. They can also be useful for studying local behavior. Figure 5, taken from Ref. 3, shows the similarity between analytically predicted and experimentally observed cracking patterns for a shear panel.

However, local models have several disadvantages. Their accuracy depends on precise multiaxial constitutive relations for all materials involved. Such relations are not available at this time for many common construction materials, particularly unit masonry. Local methods, while a valuable research tool, are too costly and time-consuming for use in analyzing the response of large, complex structures.

ANALYSES USING MACROSCOPIC MODELS

Macroscopic models are useful in analyzing large structures which would otherwise require prohibitive amounts of storage if analyzed microscopically. However, their accuracy is limited by the extent to which they duplicate actual behavior rather than simply fitting an experimental curve. Macroscopic models can generally be used most successfully in idealizing the behavior of elements whose basic models of structural resistance are well known, and which can be designed to respond stably in those resistance modes. In addition to predicting a wide range of experimental behavior, a theoretically sound macroscopic model can give information about the relative importance of various local mechanisms, thus aiding in the improvement of local idealizations.

One specific example of the usefulness of macroscopic models is their potential application to complex, infilled frame structures. Reference 8 discusses a design approach producing engineered infilled frames with stable lateral force-deflection characteristics in the elastic and inelastic ranges. The effectiveness of this design approach was investigated in a series of quasi-static, cyclic load tests on multistory infilled frame subassemblages (Fig. 6). Figures 7-9, taken from Ref. 8, show the lateral load-deflection behavior of those subassemblages. The engineered infilled frames were found to exhibit high strength, stiffness, and energy dissipation capacity. As discussed in that reference, those desirable characteristics were obtained principally by special design for column shear resistance in excess of the infill cracking shear, and should not be attributed to ordinary infilled frames. It was decided to idealize those infilled frames using macroscopic models. The models described herein were developed for use with ANSR-I, a general purpose, nonlinear structural analysis program (11).

Experimental results indicated that engineered infilled frames behaved essentially as a combination of two types of structural components: (1) the frame members themselves; and (2) the infills, which strengthened the frames, stiffened them, and dissipated large amounts of energy through distributed cracking. The presence of the engineered infills changed the basic behavior from that of a bare frame to that of a frame braced by equivalent diagonal compression struts. The process of infill panel degradation greatly influenced the location of critical regions in the frame members, and, consequently, the final mechanism of the infilled frame subassemblage. However, the forces induced by the

infills did not significantly reduce the available rotational ductility of the critical regions under cycles of reversed loading.

It was decided that the analytical model of the subassemblage should predict the following aspects of the filled frame behavior:

- (1) Initial stiffness
- (2) Initial strength
- (3) Degrading stiffness and strength behavior, particularly the pinching effect associated with the deterioration of infill stiffness.

It was considered convenient to model the infilled frame subassemblage using two separate types of elements:

(1) The frame elements were modeled using two-component elements, each consisting of a linear elastic member in parallel with an elasto-plastic one (14). Single elements were used for the columns. The beams were modeled using several parallel two-component elements to give a combined moment-rotation curve consistent with the nonlinear moment-curvature curves of each beam.

(2) As shown in Fig. 10, a pair of equivalent diagonal strut elements was used to idealize each infill panel. Three different equivalent strut models were developed. Each successive model involved a slight increase in complexity, and produced results more closely approximating those observed experimentally.

Strut Model #1. It was hypothesized that the strut model should duplicate the following main aspects of the experimentally observed infill behavior:

- (1) Initial stiffness and strength
- (2) Decreased strength with increased deformation
- (3) Decreased stiffness on reloading

A macroscopic equivalent strut element was written with the mechanical characteristics shown in Fig. 11. Note that the complete infilled frame response combined the behavior of the frame members and two equivalent diagonal struts per panel, one in each direction. The following behavior description refers to a single strut only:

- (1) Elastic Loading (path OA)

This is defined by

$$S = \frac{EA}{L} v$$

where S is the axial force in the strut; E is Young's modulus for the infill material; v is the axial deformation in the strut, positive values corresponding to extension; L is the length of the strut; taken here as the distance between diagonally opposite

nodes; and A is the product of the panel thickness and the effective width of the strut.

(2) Strength Envelope Curve (path AB)

This curve is defined by

$$S = Af_c(e^{\gamma v})$$

where A , S , and v are defined as above, and f_c is the compressive strength as determined from prism tests. The strength degradation parameter, γ , is selected on the basis of experience. A value of 1.0 was used in all analyses described here. The envelope curve was defined by a decaying exponential because this was the simplest class of mathematical expressions reflecting the desired characteristics of decreasing strength with increased deformation. It is probable that some increase in accuracy could be achieved by defining the strength envelope curve in terms of more complicated classes of functions.

(3) Elastic Unloading (path BC)

In this range, the strut unloads elastically, with a stiffness equal to the elastic loading stiffness of path OA.

(4) Tension Curve (path CC'D)

Initially, an actual equivalent diagonal strut has some tensile resistance, due to the tensile strength of the panel material (usually very low) and the action of the panel steel (also low). Tensile cracking of the strut causes this tensile resistance to drop immediately. The remaining tensile resistance is due to the action of the panel steel alone. In developing Strut Model #1, it was decided that the complexity necessary to model this drop in tensile resistance was not justified in view of the generally minor effects of infill tensile strength. The idealized tension curve was defined by

$$S = Af_t$$

where S and A are defined as above, and f_t is a constant nominal resistance whose value is based on the observed tensile resistance of the panel reinforcement. The strut models permit specification of arbitrary values of f_t . However, panel tensile resistance was not observed to have any significant effect on the behavior of the experimental models, and f_t was therefore assigned a zero value in all the analyses described herein.

(5) Reloading Curve (path DE)

The experimental models were observed to exhibit decreased stiffness upon reloading. Therefore, this reloading curve was defined by a straight line connecting the point on the tension curve corresponding to maximum positive deformation

(point D), with the point on the strength envelope curve corresponding to maximum negative deformation (point B).

(6) Further Cycles (path EF GG' E)

Strut Model #1 was defined to exhibit elastic loading and unloading during further cycles within the area defined by the strength envelope curve, the tension curve, and the reloading curve. For example, the strut unloads elastically from point E until reaching the tension curve at point F. Decreasing deformation causes movement along the tension curve from point F to point G. Reloading in compression causes the strut to reload elastically until reaching the previously defined reloading curve at point G' . The strut then continues to reload along this curve until reaching point E again. Strut extensions beyond the deformation corresponding to point D (for instance, to point D'), cause the reloading curve to be redefined in terms of the coordinates of points B and D' . A similar redefinition takes place following compressive deformations in excess of the value corresponding to point B.

Using this Strut Model #1, the entire infilled frame sub-assembly was modeled as shown in Fig. 12. This model comprised 46 elements, 18 nodes, and 40 degrees of freedom. The experimental results from all three infilled frame tests were qualitatively similar with respect to strength and stiffness degradation. It was decided to compare the analytical results with those obtained experimentally in Test #2, carried out on the clay-infilled bare frame. That test used a tip displacement history consisting of reversed cycles to increasing maximum amplitudes of approximately equal magnitude in each direction, and was therefore judged slightly more convenient for analytical comparison purposes than the other two tests, whose displacement histories were skewed in one direction. The following material parameters were used for the equivalent strut elements:

$E = 8290 \text{ MPa (1200 ksi)}$, as determined from prism tests on clay blocks loaded perpendicular to the bed joints

$f_c = 24.1 \text{ MPa (3500 psi)}$, as determined from prism tests on clay blocks loaded perpendicular to the bed joints

$f_t = 0.0$, as explained previously

$A = 12900 \text{ mm}^2 (20 \text{ in.}^2)$. This figure was obtained by multiplying the nominal thickness of the panel (51 mm, or 2 in.) times the equivalent strut width (Appendix I and Ref. 10)

$\gamma = 1.0$, as explained previously

The entire model was subjected to a cycle of reversed loading, and the results shown in Fig. 13 were obtained. Because the analytical idealization was designed to exhibit decreasing

strength for increasing deformations (beyond those required for panel cracking), a program of applied loads would clearly result in overall instability of the model after panel cracking. The ANSR-I computer program did not allow the model to be loaded by a program of specified tip displacements. To simulate displacement controlled loading, the structure was laterally restrained by a relatively stiff horizontal spring at the point where lateral loads were applied. In the analytical procedure, therefore, the applied load was taken primarily by the spring and partly by the structure. Lateral instability of the structure decreased the lateral stiffness of the spring-structure assemblage, but the combination remained stable. In Figs. 13, 15, and 18, the lateral force H is the load resisted by the structure alone, obtained in the analytical procedure by subtracting the resisting spring force from the total applied load.

Figure 13 also shows the corresponding experimentally obtained load-deflection curves for Test #2. Comparison of these two curves shows that use of Strut Model #1 produced a good representation of the experimentally observed initial stiffness and strength. However, the degrading behavior was not modeled correctly. In the analytical procedure, panel damage from post-cracking excursions in one direction did not reduce the maximum panel resistance available in the opposite direction.

Strut Model #2. To correct the above-noted deficiency of Strut Model #1, this model was refined to exhibit the mechanical behavior shown in Fig. 14 and described below:

- (1) through (4) same as Strut Model #1 (path OABCC'D)
- (5) Reloading Curve (path DB or D'B')

Strut Model #2 defines the reloading curve in a manner slightly different from that of Strut Model #1, to reflect more accurately the effect of previous damage history on panel strength and stiffness. Experimentally, it was observed that after reaching a given resistance level in one direction, an infilled frame model was not able to develop more than this resistance in the other direction upon load reversal. Physically, this can be explained by the fact that the two equivalent diagonal compression struts share the portion of material at the center of the panel and are therefore not independent. When a single panel infilled frame, whose panel is idealized by two equivalent struts, is loaded laterally in the positive direction, one strut will be placed in compression, and the other one will be placed in tension. The compression strut will load elastically, reach the strength envelope curve, and suffer increasing damage as it moves along the path AB of Fig. 14. The tension strut will offer some nominal tensile resistance, and will intersect and move along the tension curve on a path such as OG'D. When the direction of the lateral load is reversed, the strut which was originally in compression will unload and go into tension along a path such as BCC'FGD. The strut which was originally in tension will reload in compression.

Experimental tests showed that the infilled frame developed, upon reversal of loading, a lateral resistance equal at most to the degraded resistance in the original direction. In terms of the strut model, this implies that the strut which was originally in tension (at point D, say) will not reload in compression directly to point A (Fig. 14), but rather to point B along a line DB. The reloading curve for a given strut should be defined to connect the point on the tension curve corresponding to that strut's maximum positive deformation (point D), with the point on the strength envelope curve corresponding to the maximum negative deformation of the opposite strut in the same panel. However, because of the way in which element data are stored during execution of the ANSR-I program, this type of behavior was very difficult to prescribe, and an alternative procedure was therefore devised: in the range of panel deformations associated with significant panel cracking, experiments showed that the most significant panel deformation was in shear. When a panel idealized by two equivalent compression struts deforms in shear, the axial deformations of the struts are equal in magnitude but opposite in sign, and the maximum negative (compressive) deformation of a given strut is equal in magnitude to the maximum positive (tensile) deformation of the other strut. Therefore, it was possible to define the reloading curve in the following manner: for the single cycle of reversed loading considered in this example, the reloading curve for a given strut should be defined to connect the point on the tension curve corresponding to that strut's maximum positive deformation (point D), with the point on the strength envelope curve corresponding to the maximum positive deformation of the same strut. This definition of the reloading curve was much easier to incorporate into the analytical model. Finally, consider extending this definition to loading programs other than the single cycle of reversal considered above: suppose an equivalent strut has been loaded onto the strength envelope curve (along path OAB, say), and then unloaded to the tension curve but without significant reversal. When that strut is reloaded, its strength will clearly not be governed by the negligible amount of damage (compressive deformation) suffered by the opposite strut. The strength of the reloaded strut will depend on the damage that it itself has suffered, i.e., on its own maximum negative (compressive) deformation. For the case of a general lateral load program, the reloading curve for a given strut was, therefore, defined to be the straight line connecting the point on the tension curve corresponding to that strut's maximum positive (tensile) deformation, with the point on the strength envelope curve corresponding to the maximum (absolute value) deformation--positive or negative--previously experienced by that same strut. Referring to Fig. 14, consider the following two examples: First, suppose that a strut has been loaded following the path OABCC'FGD. Because the maximum compressive deformation (point B) is greater in magnitude than the maximum tensile deformation (point D), the former will govern, and the strut will have a reloading curve defined by the straight line DB. Physically, this would represent a case in which the panel had been subjected to some load reversal, but not enough to damage the opposite strut more than the strut under consideration.

Therefore, the damage in the reloading strut (a function of the maximum compressive deformation of that strut) would control. Second, suppose that a strut has been loaded following the path $OABCC'FGDD'$. Because the maximum compressive deformation (point B) is smaller in magnitude than the maximum tensile deformation (point D'), the latter will now govern, and the strut will have a reloading curve defined by the straight line $D'B'$, where B' and D' are located at equal distances but opposite directions from the vertical axis of Fig. 14. Physically, this would represent a case in which the panel had been subjected to severe load reversal, damaging the opposite strut. The damage to that opposite strut would control. Such damage would be a function of the maximum compressive deformation of that opposite strut, which in turn would be essentially equal (as explained above) to the maximum tensile deformation of the strut under consideration.

(6) Further Cycles (path $EFGG''E$)

Strut Model #2 was defined identically to Strut Model #1 in this range. Because of the change in definition of the reloading curve between Strut Models #1 and #2, however, redefinitions of the reloading curve could occur following increases in maximum strut deformation in either sense. For example, referring to Fig. 14, the reloading curve DB would be redefined after strut deformations along the path DBB' , or after strut deformations along the path GDD' .

With the same element properties and loading program as before, the use of Strut Model #2 produced the results shown in Fig. 15. That same figure also shows the experimental behavior observed in Test #2. Because only the reloading curve had been changed from Strut Model #1 to Strut Model #2, the analytically predicted initial strength and stiffness continued to agree well with the experimental results. Strength degradation under monotonic load was also reproduced well. However, it may be seen that upon reversal of loading, Strut Model #2 did not produce the observed pinching effect associated with the opening of cracks in the panels.

Strut Model #3. To correct this deficiency, it was decided to introduce some additional refinements into the reloading behavior of Strut Model #2. As discussed in the previous subsection, this strut model exhibited linear reloading behavior. Actually, experimental observations showed that reloading behavior consisted of two distinct phases. In the first phase, the previously formed vertical panel cracks close. Until this closure is complete, the panel's lateral strength and stiffness are essentially zero. Closure occurs when the panel is returned to its undeformed configuration (or, in terms of the equivalent strut idealization, when deformations in the equivalent struts are zero). In the second phase, following panel crack closure, the panel reloads, but with reduced stiffness and strength compared to the virgin elastic behavior. In terms of the equivalent strut

idealization, the compression strut reloads with reduced stiffness and strength.

In accordance with the above experimental observations, the reloading behavior of Strut Model #2 was slightly modified to produce a new model, Strut Model #3. This model exhibited the mechanical behavior which is shown in Fig. 16 and described below:

(1) through (4) same as for Strut Models #1 and #2 (path OABCC'D)

(5) Elastic Unloading (path DE or D'E')

Equivalent strut tensile resistance is due primarily to the action of the panel steel. As an equivalent strut with open tension cracks begins to reload in compression, the steel crossing those cracks will experience decreased tensile stresses. This effect is not believed to significantly affect the panel's mechanical behavior, which is dominated at this stage by the opposite strut. This elastic unloading stiffness was defined to be identical to the initial elastic stiffness (path OA). It is believed that this definition, though arbitrary, is not unreasonable.

(6) Reloading Curve #1 (path EO or E'O)

This curve corresponds to the first phase of reloading discussed above. Reloading is characterized by zero stiffness and strength until the equivalent strut deformation returns to zero.

(7) Reloading Curve #2 (path OB or OB')

This curve corresponds to the second phase of reloading discussed above. Reloading is characterized by reduced stiffness and strength compared to the virgin elastic behavior of path OA. Reloading curve #2 of this strut model is defined almost exactly as for Strut Model #2: for a given strut, reloading curve #2 is the straight line connecting point 0 with the point on the strength envelope curve corresponding to the maximum (absolute value) deformation--positive or negative--previously experienced by that same strut. The previous examples apply almost verbatim to this case: if damage in the reloading strut controls, then reloading curve #2 will be defined from point 0 to point B; if damage to the opposite strut controls, the curve will be defined from point 0 to point B', where B' and D' are located at equal distances but opposite directions from the vertical axis of Fig. 16.

(8) Further Cycles (path FGHH'F)

Strut Model #3 is identical to Strut Model #2 in this range. As with Strut Model #2, redefinition of reloading curve #2 would occur following increases in maximum strut deformation in either sense. For example, referring to Fig. 16, reloading

curve #2 (path OB) would be redefined after strut deformations along the path OBB', or after strut deformations along the path HDD'.

The complete idealized subassembly was subjected to a displacement history (shown in Fig. 17) similar to that used experimentally. As noted previously, deflection control was simulated by the use of a stiff lateral restraining spring. Figure 18 shows the analytical results, together with the experimental behavior observed for Test #2. Comparison of these two curves shows that the use of Strut Model #3 produces a good representation of the experimentally observed stiffness, strength, and degradation characteristics of the entire infilled frame subassembly, throughout a wide range of load and displacement reversals.

It should be noted that Strut Model #3 produces an analytical subassembly model which will not in general exhibit the experimentally observed phenomenon of decreasing lateral resistance with cycles of full displacement reversal to constant maximum amplitude. Also, the analytically predicted decrease in load following initial panel crushing is in some cases slightly less than that observed experimentally. As noted previously, Strut Model #3 could be modified to reflect this by introducing slight refinements in the envelope curve.

However, it is believed that the procedure discussed here can be used in analyzing the response of large, engineered infilled frame structures. For example, the entire subassembly discussed herein was analyzed using a model comprising 18 nodes, 40 degrees of freedom, and 46 members. Approximately 100 load steps were required to compute the response throughout the more than three cycles of full displacement reversal. All operations were performed in core, and required 7.0 minutes of central processor time on the CDC-6400.

CONCLUSIONS

Both local and macroscopic approaches have been found useful for analyzing the nonlinear response of structural components. The two approaches are complementary. Local idealization can be used to develop more accurate macroscopic models, while macroscopic models can be used to assess the relative importance of various local mechanisms. Based on the results presented herein, it is believed that macroscopic idealizations are efficient and practical for analyzing the effects of nonstructural elements on the elastic and inelastic response of large, complex structures.

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APPENDIX I. EQUIVALENT STRUT WIDTHS

Equivalent strut widths have been determined by many investigators to depend principally on the panel aspect ratio and the ratio of panel stiffness to frame stiffness. Widths typically vary from about one-third to one-eighth of the length of the panel diagonal. The formulas used herein are taken from Ref. 10:

$$\lambda_h h = \sqrt[4]{\frac{E' t \sin 2\theta}{4EI_h h'}}$$

and

$$\left(\frac{w'_{ek}}{w}\right) = 0.175 (\lambda_h h)^{-0.4}, \text{ units in inches}$$

where

h, h', θ , and w' refer to Fig. 19

w'_{ek} is the effective strut width for stiffness

E' is the panel modulus

E is the frame modulus

I_h is the column moment of inertia

t is the panel thickness

In our case,

$h = 32$ in. (0.813 m) for the bottom panel, and
 36 in. (0.914 m) for the others

$E = 3600$ ksi (24822 MPa)

$t = 2$ in. (51 mm) nominal

$$E' = 1200 \text{ ksi (8274 MPa)}$$

$$I_h = 135.16 \text{ in.}^4 \text{ (} 5.62 \times 10^{-5} \text{ m}^4 \text{) gross transformed section}$$

$$h' = 28 \text{ in. (0.711 m)}$$

so that

$$\lambda_h h = 2.35 \text{ in. (60 mm) for the bottom panel, and} \\ 2.65 \text{ in. (67 mm) for the others}$$

since

$$w' = (28)^2 + (74)^2 = 79.12 \text{ in. (2.00 m)}$$

$$w'_{ek} = 9.83 \text{ in. (250 mm) for the bottom panel and} \\ 9.38 \text{ in. (238 mm) for the others}$$

Because the equivalent strut width of the lower panel is close to the widths of the other two panels, it was decided to use the average value of 9.61 in. (244 mm) for all struts. In the author's opinion, the considerable scatter in the data points used to determine the empirical formulas of Ref. 10 does not justify the use of a strut width more precise than 10 in., and it was decided to use this nominal figure. The error between 9.61 in. (244 mm) and 10 in. (254 mm) is less than 5 percent, and the use of the former figure was believed to imply an unwarranted precision. The strut area was then calculated to be

$$2 \text{ in.} \times 10 \text{ in.} = 20 \text{ in.}^2 \text{ (0.26 m}^2\text{)}$$

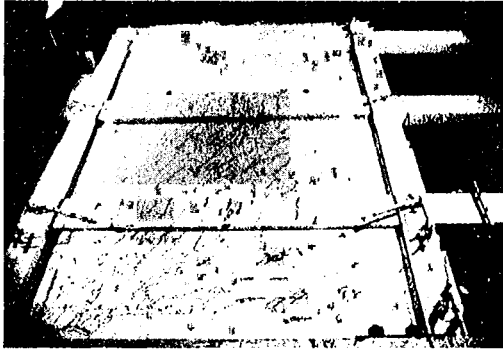


Fig. 1--Development of equivalent struts



Fig. 2--Crushing of equivalent strut



Fig. 3--Formation of sidesway mechanism

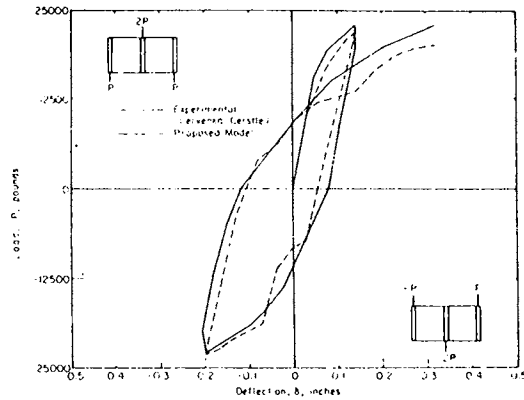


Fig. 4--Comparison of local model to experimental curve

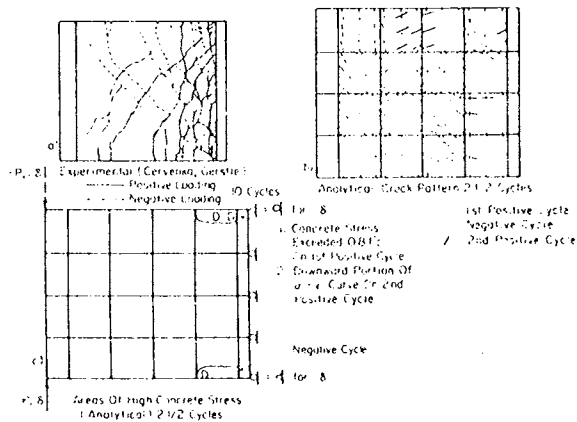


Fig. 5--Comparison of analytical and experimental crack patterns

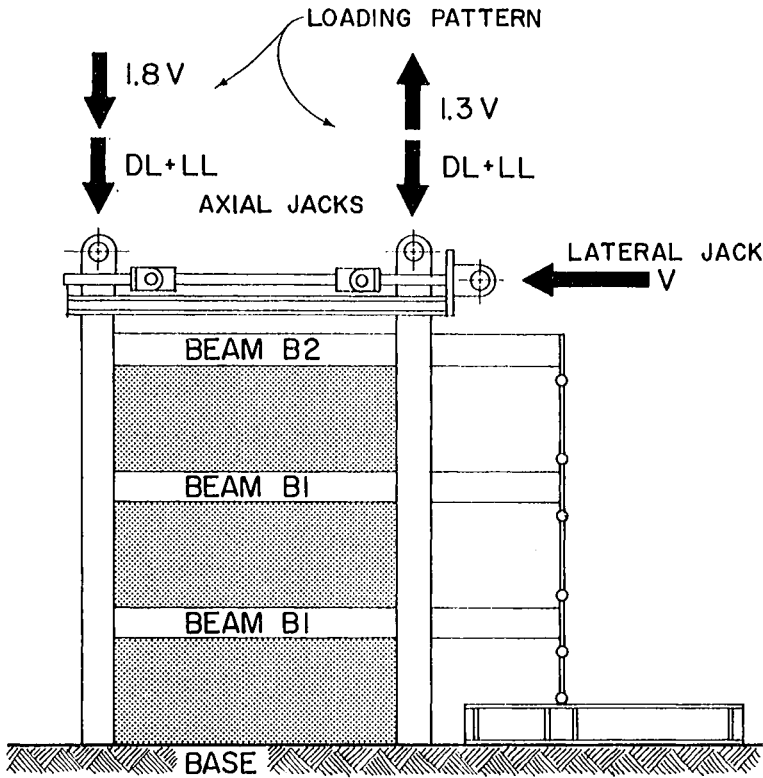
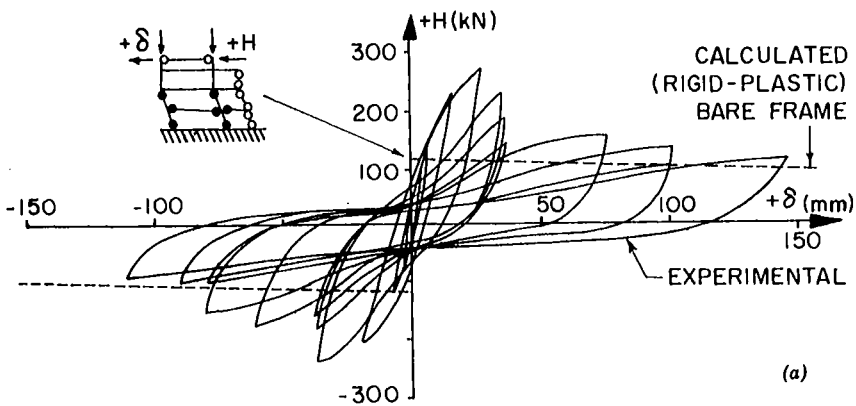


Fig. 6--Infilled frame specimen



(a)

Fig. 7--Lateral load-deflection relationship--Clay-infilled bare frame (Test 2)

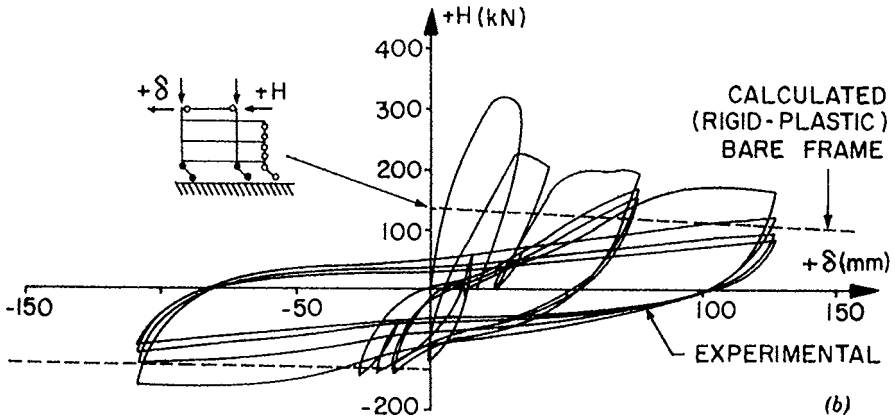


Fig. 8--Lateral load-deflection relationship--Virgin frame, clay infill (Test 3)

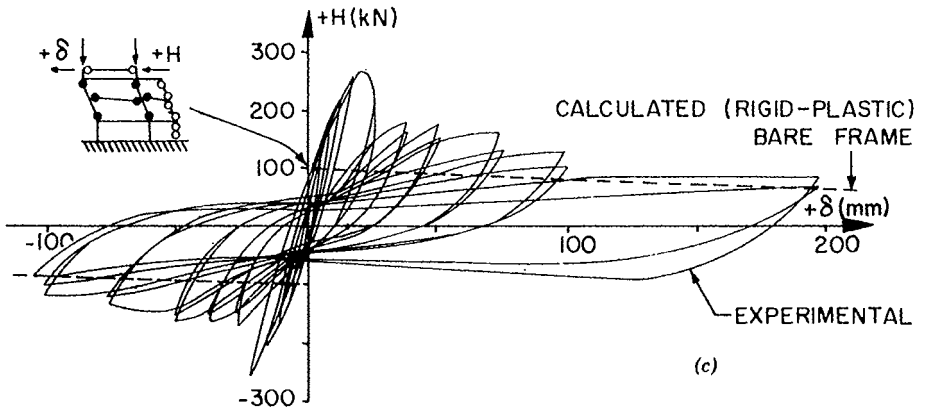


Fig. 9--Lateral load-deflection relationship--Virgin frame, concrete block infill (Test 4)

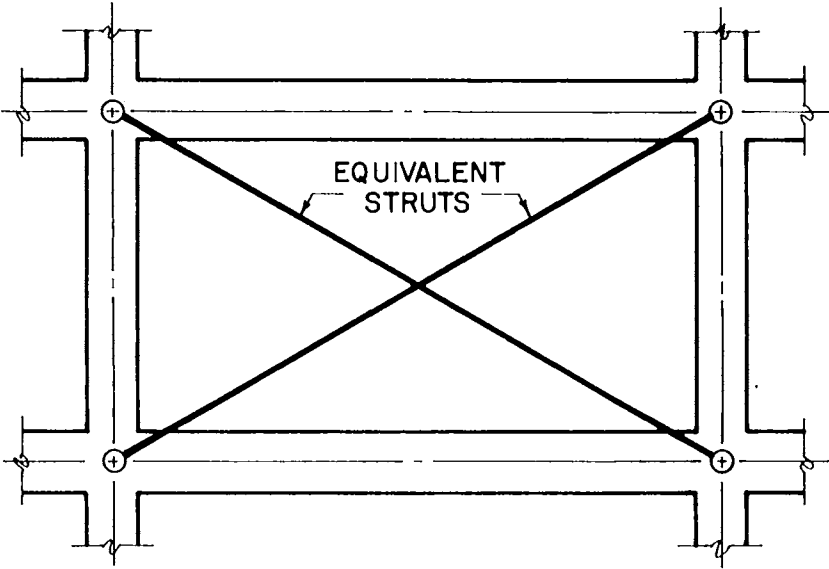


Fig. 10--Analytical idealization of infill panel as two equivalent struts

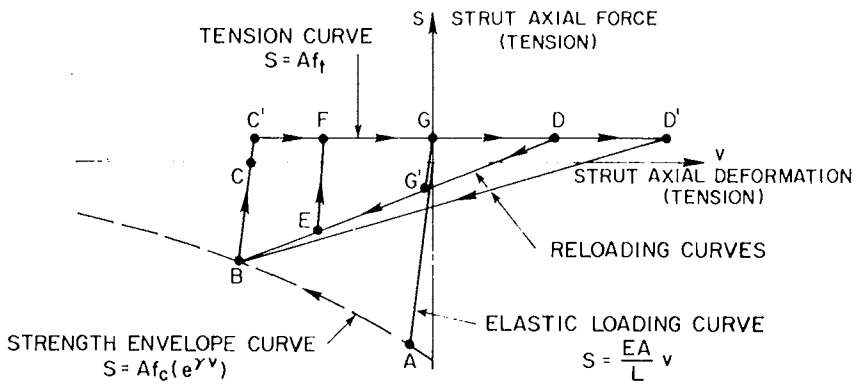


Fig. 11--Mechanical behavior of Strut Model 1

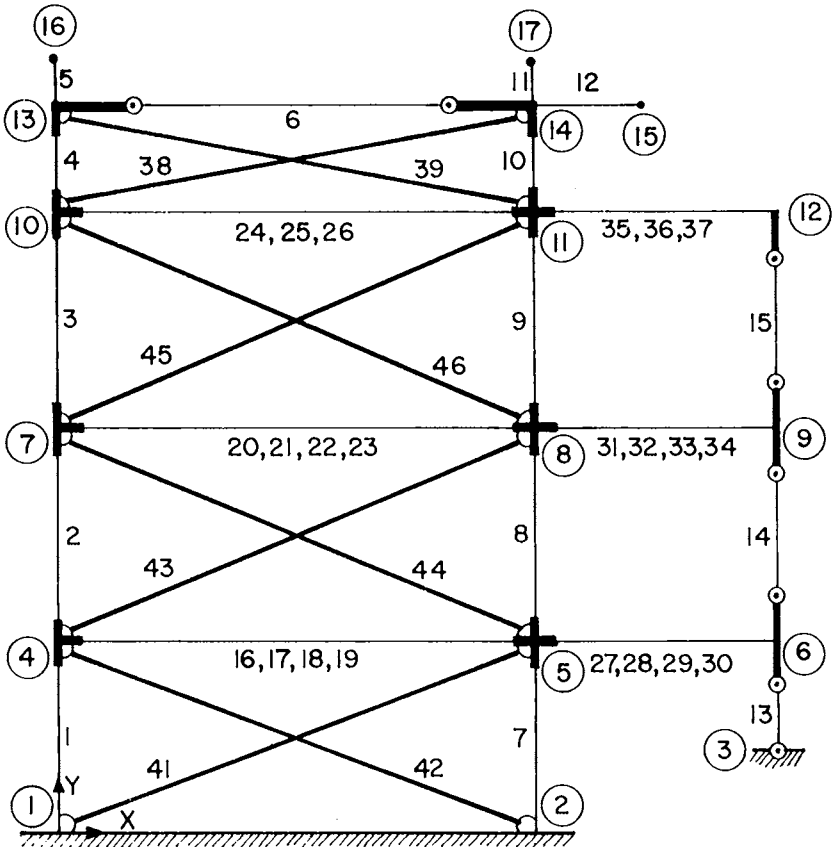


Fig. 12--Analytical idealization of infilled frame subassemblage

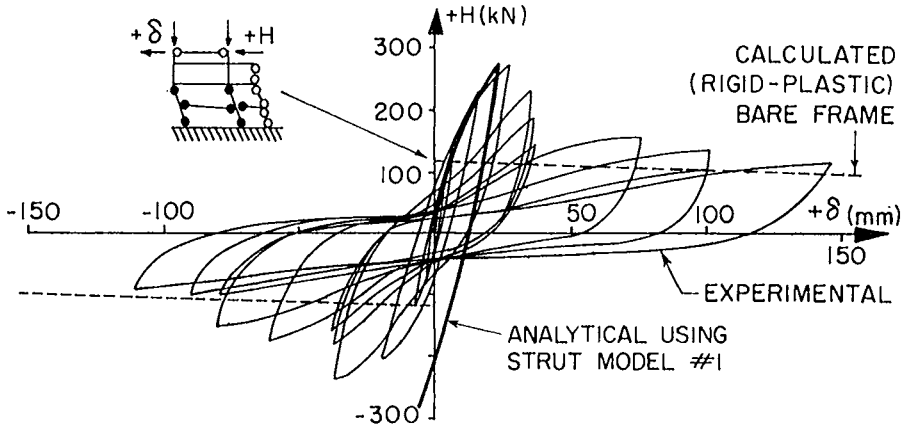


Fig. 13--Comparison of experimental response (Test 2) with analytical prediction--Strut Model 1

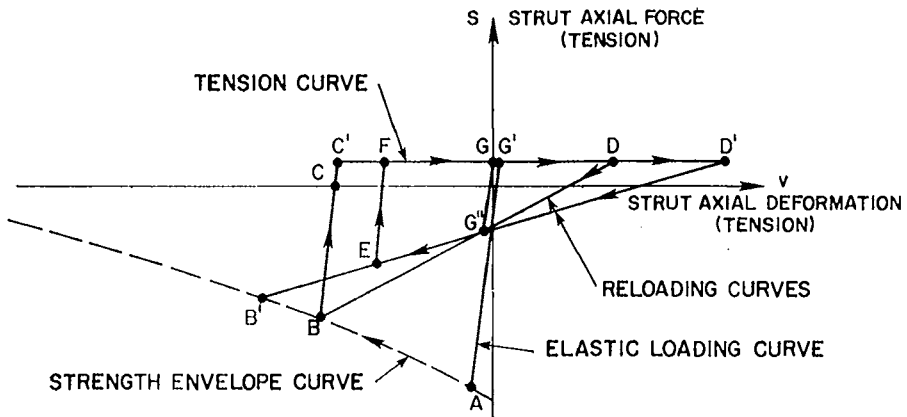


Fig. 14--Mechanical behavior of Strut Model 2

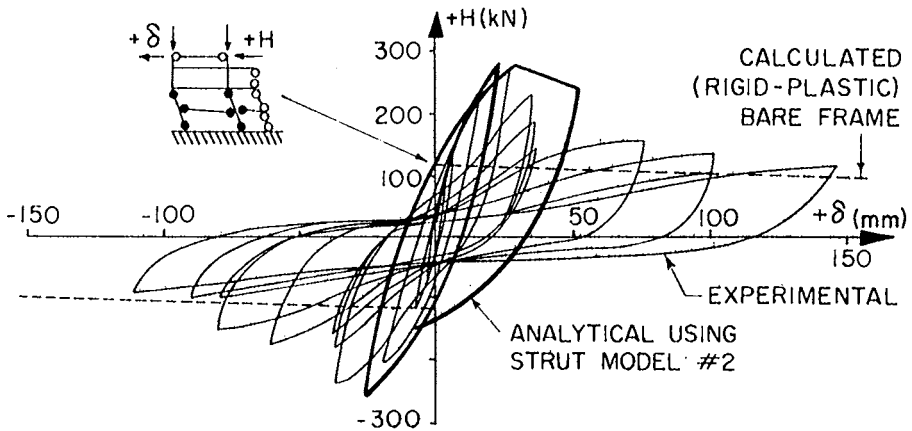


Fig. 15-- Comparison of experimental response (Test 2) with analytical prediction--Strut Model 2

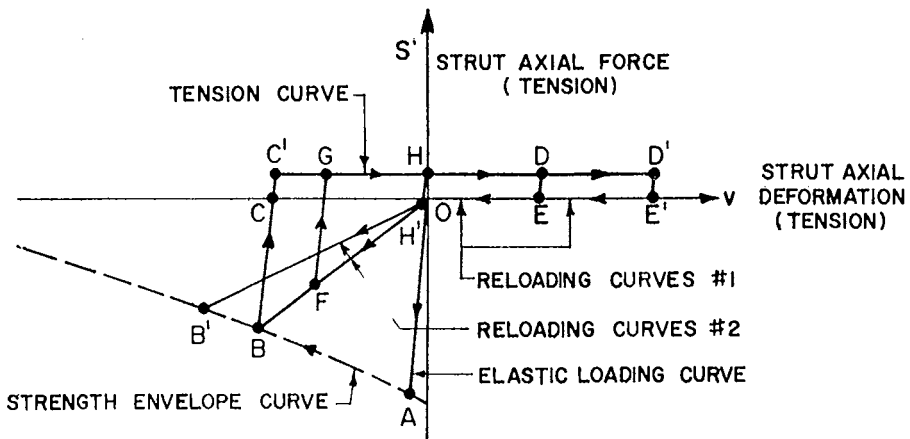


Fig. 16--Mechanical behavior of Strut Model 3

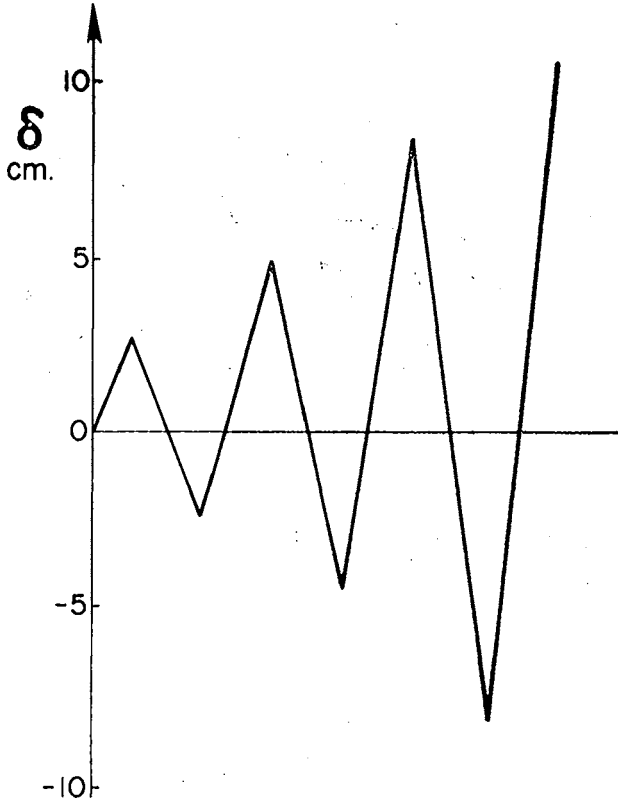


Fig. 17--Loading program used for analytical prediction of response

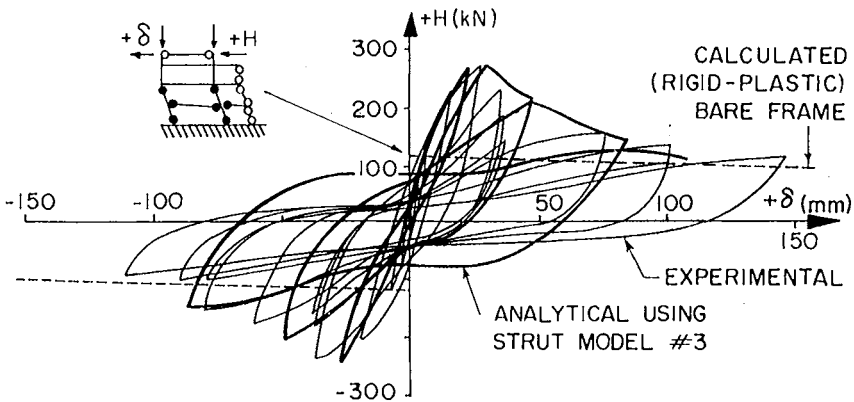


Fig. 18--Comparison of experimental response (Test 2) with analytical prediction--Strut Model 3

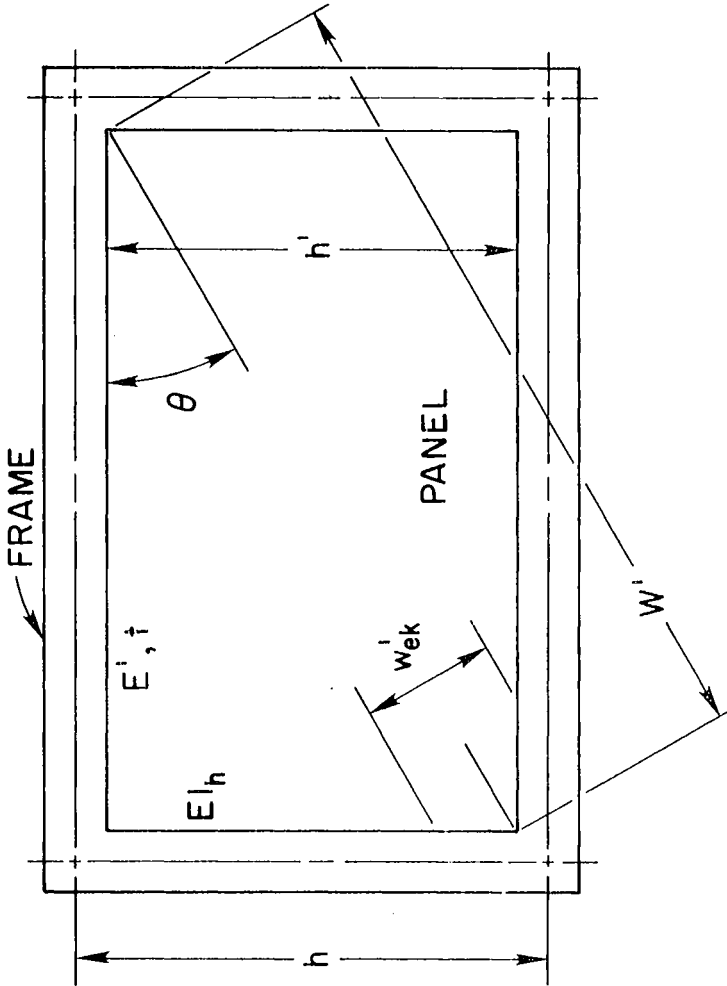


Fig. 19--Nomenclature used in Appendix I

Effect of Nonlinear-Inelastic Connection Behavior on Precast Panelized Shearwalls

By C. Llorente, J. M. Becker, and J. M. Roesset

Synopsis: This paper reports on preliminary results of research into the seismic response of large panel precast concrete construction typical of that used in the United States. Seismic response is being modeled using finite element techniques and an explicit time step integration procedure. The seismic resisting elements of a crosswall system have been isolated to study the role the connection regions may have in seismic response. Results are presented that show the effects associated with rocking type motion and slippage in the horizontal connections of isolated crosswalls.

Keywords: concrete panels; connections; dynamic loads; earthquake resistant structures; lateral pressure; loads (forces); mathematical models; precast concrete; research; shearwalls; slippage.

C. Llorente is working towards a Ph.D. in Civil Engineering at the Massachusetts Institute of Technology, where he is currently a research assistant.

J. M. Becker is an Associate Professor of Civil Engineering at the Massachusetts Institute of Technology. He obtained a Ph.D. from the Univ. of Calif., Berkeley, in 1973. Dr. Becker is a member of the ACI, ASCE and ASHRAE. His recent research has focused on the seismic resistance of precast concrete panel buildings.

J. M. Roeset, formerly a Professor at the Massachusetts Institute of Technology, is now a Professor in Civil Engineering at the Univ. of Texas, Austin. He received a Sc.D. from M.I.T. in 1964. Dr. Roeset is a member of the ASCE and ACI. His research interests include behavior of structures under dynamic loads.

INTRODUCTION

In large panel precast concrete buildings, resistance to lateral loads often depends upon isolated or simple loadbearing crosswalls. A typical situation that leads to this dependency is illustrated in Figure 1. The seismic response of such a wall assemblage is highly influenced by the behavior associated with their connections (1,9). These connections introduce planes of weakness along which the nonlinear inelastic behavior of the system will concentrate.

Analysis procedures accounting for this connection dependent behavior have been reported by Brankov and Sachanski (2) and Powell and Schricker (7). Brankov's work modeled the slippage associated with shear friction force transfer, while Powell has combined both the shear slippage and connection opening (a rocking type motion). In both cases the connection characteristics were lumped into single connection parameters. This paper extends these approaches into a more generalized connection model that is capable of handling both rocking and slippage while considering the actual stress distribution across the entire connection region.

The modeling work to be presented has been carried out using the large panel structure illustrated in Figure 1 as an example. In this system prestressed precast hollow core planks span between adjacent crosswalls. Vertical continuity is provided in the platform-type horizontal connection (see Figure 2) by post-tensioning, which is achieved through the use of ungrouted bars of high strength steel. The horizontal edges of the panels are plain, so that shear transfer in the connection is primarily through friction.

APPROACH TO MODELING

The dynamic behavior of large panel precast concrete buildings can be quite complex, and the development of dynamic models that can represent all aspects of behavior is rather difficult if not impossible. To reduce the size of the problem, simplifying assumptions are introduced that help isolate the most important behavioral aspects of these buildings. Some of these simplifying assumptions, as well as the procedures used to model the wall panels and connection behavior, will be treated in this section.

A finite element idealization is used as the basis for modeling the isolated crosswall. This technique has seen extensive use in the analysis of large panel structures for both static loads (5,6) and more recently for seismic considerations (3,4). The resulting dynamic equilibrium equations are integrated using the Central Difference Method.

Modeling Assumptions

The studies reported herein have been carried out on a building similar to that shown on Figure 1. The basic assumptions made with regard to the dynamic behavior of the building studied are the following:

- In the transverse direction the coplanar crosswalls are linked together by a lintel with no effective bending stiffness; thus, all studies of response in this direction have been carried out on an uncoupled, isolated wall.
- The floor system is assumed to act as a rigid diaphragm, so that the mass on any given floor level is distributed among all walls in a given direction according to their stiffness.
- The foundation is assumed to be rigid.
- Because the connection regions are planes of diminished strength and stiffness, the nonlinear and inelastic behavior in large panel constructions may be isolated in the connection regions. Thus wall panels can be modeled as linear elastic components. (Note: This assumption should be restricted to solid wall panels because penetrated panels can exhibit inelastic behavior in the lintels and piers.)
- The post-tensioning bars are assumed to be ungrouted, and so they are modeled as truss elements connected from the base to any desired floor level.

It is recognized that, in terms of the response of the over-all structure, these assumptions may not be entirely justifiable; it is felt, however, that they would allow for an in-depth examination of basic behavioral mechanisms.

Modeling of Wall Panels

Because the wall panels are to remain linear elastic, they were modeled using a substructuring technique. In this approach, the degrees of freedom of a wall panel, which are not needed to interconnect it to the other components, are eliminated through static condensation. The resulting elements, known as substructures, can be assembled in a manner directly analogous to the actual structure. This procedure has been used extensively by Petersson (6) in his work on progressive collapse. Frank (3) has also employed this technique in modeling the seismic response of panelized structures.

Modeling of Pure Rocking Motion

The continual opening and closing that occurs along the connection interface (between the upper wall panel and the floor plank on which it rests) under the action of lateral loads, is termed pure rocking motion. This opening will occur whenever the axial stresses caused by the lateral loads (earthquake or wind) exceed the compressive stresses due to the gravity loads and post-tensioning. Once opening has occurred, in the post-tensioned connection, there will be no resistance to further opening, thus the tensile stiffness of the connection will be zero.

This behavior was modeled using four node plane stress rectangular elements with no tension material properties. Thus the rigidity matrix of the elements are given by the following:

$$C = \frac{E}{1-\nu^2} \begin{pmatrix} 1 & \nu & 0 \\ \nu & 1 & 0 \\ 0 & 0 & \frac{(1-\nu)}{2} \end{pmatrix} \quad \text{for } \epsilon_y \leq 0 \quad (1)$$

$$C = 0 \quad , \quad \text{for } \epsilon_y > 0$$

where: C = element rigidity matrix
 E = Young's Modulus
 ν = Poisson's ratio
 ϵ_y = axial strain

In order to allow opening to occur gradually, the tangent stiffness matrix of the element was calculated as follows:

$$k = \int_V B^T C B dV = b \left[\int_{A_c} B^T C B dA + \int_{A_t} B^T C B dA \right] = b \int_{A_c} B^T C B dA \quad (2)$$

where: k = tangent stiffness matrix of connection element
 A_t, A_c = area of element which is under tension and compression respectively
 B = strain-displacement transformation matrix
 b = connector thickness

A closed form solution for the integral on A_c is possible, and the resulting stiffness matrix can be found in Reference 4.

Modeling of Rocking with Shear Slip

Shear transfer in horizontal connections of the type being studied (Fig. 2) is primarily accomplished through shear friction. Experimental work on shear transfer in horizontal connections (8) suggests a bilinear shear stress-shear deformation relationship for a constant level of compressive stress (Fig. 3). The initial elastic portion of the curve represents the relative shear displacement Δu_d caused by shear distortion in the connection material, while the second slope, G_s , is associated with the slippage Δu_s along the interface between the wall panel and the floor plank. The material model used for the axial behavior of the connection is shown in Figure 4.

The connection regions were discretized using four-node rectangular elements. Figure 5 shows a portion of the discretized connection region. The axial and shear deformations are defined as for a "bond or interface" element on the assumption of independent, uncoupled distributed springs. Thus,

$$\begin{Bmatrix} \Delta v \\ \Delta u \end{Bmatrix} = B U \quad (3)$$

where

$$B = \frac{1}{2} \begin{bmatrix} 0 & (1+r) & 0 & (1-r) & 0 & -(1-r) & 0 & -(1+r) \\ (1+r) & 0 & (1-r) & 0 & -(1-r) & 0 & -(1+r) & 0 \end{bmatrix} \quad (4)$$

$$U^T = [u_1 \ v_1 \ u_2 \ v_2 \ u_3 \ v_3 \ u_4 \ v_4] \quad (5)$$

The restoring nodal forces $F_r(t)$ for a typical element are evaluated as follows:

$$F_r(t) = \frac{1}{2} b L \int_{-1}^{+1} B^T \begin{Bmatrix} \sigma(t) \\ \tau(t) \end{Bmatrix} dr \quad (6)$$

and the stresses are given by

$$\begin{aligned} \sigma(t) &= \sigma(t-\Delta t) + \Delta \sigma \\ \tau(t) &= \tau(t-\Delta t) + \Delta \tau \end{aligned} \quad (7)$$

where: $\sigma(t)$ = axial stress at time t

$\tau(t)$ = shear stress at time t

$\Delta \sigma$ = increment in axial stress during time interval Δt

$\Delta \tau$ = increment in shear stress during time interval Δt

b = thickness

The nodal restoring forces $F_r(t)$ were evaluated numerically using two or more integration points along the r direction (see Figure 5).

In an actual structure, the compressive stress across the connection is not constant, but varies with the level of overturning moment, which means that the level at which slip will occur will not remain constant. In order to account for the variation in compressive stress in the calculation of Δt it was assumed that the compressive stress was constant during a time interval Δt thus allowing for the use of the basic shear stress-shear deformation relationship. Furthermore it was assumed that when the shear strength dropped, the shear stress would unload elastically to the new level of shear strength, and then slip would take place. Figure 6 shows the shear stress-shear deformation behavior at some integration point along the connection where the axial stress fluctuates as shown. It should be added that unloading will take place elastically, and no slippage will occur regardless of the value of axial stress until the curve crosses the $\tau = 0$ axis.

SEISMIC RESPONSE OF ISOLATED WALL

Since actual earthquake and experimental information on precast panel construction is limited, especially for the systems being built in the U.S., it becomes necessary for the researcher and designer to turn to analytical procedures to develop a generalized sense of seismic behavior. This section will present results of analytical studies pertaining to American panelized construction, that may help highlight some of the important behavioral aspects of these systems. All results presented in this paper are for a ten story precast shear wall. The wall dimensions and finite element discretization are given in Figure 7, while the gravity loads and masses and material properties are given in Appendix 2.

In all the analysis, use was made of an artificial earthquake generated to match the Newmark-Blume-Kapur response spectrum for 2% of critical damping. Figure 8 shows the response spectrum of the artificial motion for 5% of critical damping. The acceleration time history is shown on Figure 9.

Response Under Pure Rocking Motion

A shear wall undergoing pure rocking motion exhibits a softening type behavior as opening progresses. This type of behavior does not cause energy dissipation, instead, it affects the natural periods of the structure. As the structure gets progressively softer, that is, its stiffness decreases, its instantaneous natural periods become longer. This elongation in periods causes shifts within the response spectrum, and depending on the nature of the response spectrum, the resulting force levels and deformations may be higher or lower.

Results typical of pure rocking motion are summarized on Figure 10. The results have been normalized with respect to the elastic response at 25% peak acceleration. As can be observed,

there is a decrease in the maximum force levels obtained. This decrease in forces can be explained by noting that any elongation in the fundamental period will cause a downward shift in the acceleration spectrum (Fig. 8) leading to lower forces. The opposite is true for deformations, where now an increase in period will only cause increases in deformations over the elastic response.

Figure 11 compares the connection axial strain and shear stress distributions at the time of maximum tensile strain. The pure rocking case exhibits high shear stress concentrations around the tip of the crack, similar to behavior observed in fracture mechanics. In an actual structure, these high shear stresses cannot be sustained without slippage occurring locally around this region, because of the low value of compressive stress. This indicates the need to incorporate slippage to account for local shear redistribution.

Response Under Rocking With Shear Slip

In this series of studies, two friction coefficients were considered, $\mu = 0.2$ and $\mu = 0.4$. Similarly, two peak ground accelerations were used, 25% and 30% of gravity. The value of $\mu = 0.4$ has been suggested by Brankov (2), and it is also typical of values obtained from tests performed at the PCA laboratories. The value of $\mu = 0.2$, although low, may represent a case where the slip surfaces have been smoothed after an earthquake has occurred. Figure 12 summarizes some of the most pertinent results obtained for the behavior under rocking with shear slip. The cases with the higher friction coefficients ($\mu = 0.4$) experienced global slip (i.e., slip occurred on all points along the connection), so their response is a combination of rocking and slip. All cases however, exhibit substantial decreases in force levels.

The shear force levels for the cases with $\mu = 0.2$ are comparable, and essentially equal to the total compressive force (gravity plus post-tensioning) times the coefficient of friction. However, the magnitudes of the shear slips and deformations attained under the higher acceleration are larger as would be expected.

There are increases in lateral deflections over the linear elastic case. However, the cases with $\mu = 0.4$ exhibit a more dramatic increase in lateral deflections. This is caused by increased cross-sectional rotations resulting from large gap openings that occurred throughout the height of the wall. Lowering the friction coefficient to $\mu = 0.2$ limited the force levels such that the size of the gap openings were decreased to more than half (8.8mm vs. 3.8mm maximum gap opening at the base for 30% peak ground acceleration).

Figure 13 shows a comparison of the axial strain and shear

stress distributions at the time of maximum gap opening. Again, the difference in gap openings between the lower and higher friction coefficient cases can be observed. Note that cracked widths on the order of 70% were obtained, leaving only a fraction of the connection region (about 30%) for force transfer. This condition leads to stress concentrations that may cause distress in the connection or wall panels. Allowing slippage relieved the high shear stress concentrations associated with the pure rocking behavior. The case with $\mu = 0.4$ shows slippage locally around the tip of the crack, while in the other case ($\mu = 0.2$) the shear strength has been exceeded along all points in the connection. Its shear distribution is therefore identical to the compressive stress distribution times μ .

Figure 14 shows the hysteretic behavior at an integration point close to the neutral axis of the wall. At such a point, the level at which slip occurs is nearly constant and equal to the initial compressive stress times μ . Points further away from the neutral axis are highly influenced by fluctuations in compressive stress so their behavior is more erratic.

To obtain a sense of the energy dissipation associated with slippage, consider the time histories of base shear shown on Figure 15. In addition to its substantial decrease in amplitude, the case with $\mu = 0.2$ shows a noticeable change in frequency content. The higher frequencies have been filtered out (damped) indicating the extent of energy dissipation.

A major concern in the use of ungrouted post-tensioning bars is the possibility of yielding occurring. Figure 16 shows the stress history of one of the post-tensioning bars. Increases in stress on the order of 20% over the initial stress were observed; however, the bars still remain well within the elastic range, indicating favorable response.

CONCLUSIONS

This paper is a brief presentation of research currently being carried out at the Massachusetts Institute of Technology on the seismic behavior of precast concrete panel buildings. The work presented herein deals with only one aspect of the total research effort, namely the influence of the horizontal connections on the dynamic behavior of an isolated precast shear wall.

This study required the development of suitable analytical models in conjunction with computer programs written specifically to handle the pure rocking motion and the rocking with slip behavior. The following are some of the most important conclusions reached with regard to the analytical models developed:

1. The two models capture the most important behavioral aspects of the horizontal connections, namely that the magnitude of the cracked widths, gap openings and the stress

distributions can be determined.

2. The models allow for gradual opening to occur. This is because transfer of forces does not occur at discrete points in a connection, but rather it is effected along the plane of the horizontal connection. This avoids problems that may arise by sudden changes in stiffness that occur when opening is monitored at discrete points.
3. Other sources of nonlinear behavior, such as joint degradation (cracking of concrete) can be easily incorporated into the models.

The limited results presented in this paper have illustrated the potential role that the connection regions in panelized construction may play in seismic response. Some of the most important favorable aspects of connection behavior are the following:

1. Rocking motion will not necessarily lead to increases in force levels, indeed, it may contribute to a lessening in the forces attained.
2. Slippage allows for some limited energy dissipation.
3. Allowing slipping in the plane of the horizontal connections may provide a reasonable mechanism for limiting seismic forces.

Figure 17 summarizes the behavior that may be expected from nonlinear-inelastic connection behavior. As can be observed, connection behavior may lead to lower forces and increased deflections.

On the other hand, the connections may also lead to unfavorable seismic response. The possible occurrence of large cracked widths cause high stress concentrations to occur along the closed portion of the connection. This may lead to distress in the connection region or wall panels as shown (Figure 18). In such a case, the magnitude of shear forces that may be transferred along this region is no longer limited by friction, but by the potential for degradation to occur in the connection or wall panel. This potential for degradation, whether it is in the connection or wall panel, must be avoided in order to insure the overall integrity of the structure.

The results presented in this paper are by no means conclusive, and further work is necessary before firm design recommendations can be made. Further research is still needed in areas such as:

- Development of a material model that includes the effect of possible degradation in the connection or wall panels.

- More extensive parametric studies should be made to determine the sensitivity of the response to different ground motions.
- Inclusion of the possible effects of soil-structure interaction and vertical ground motion.
- And perhaps most important, experimental work is needed that may provide a better understanding of the dynamic behavior of these structures, and as a means of calibrating some of the analytical models being developed.

ACKNOWLEDGEMENTS

This research on the Seismic Resistance of Precast Concrete Panel Buildings is being conducted at the Massachusetts Institute of Technology and is sponsored by the National Science Foundation under Grant ENV 75-03778.

APPENDIX 1 - REFERENCES

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APPENDIX 2 -- MODELING PARAMETERS

Wall Panels:	$E = 25.2 \text{ GPa}$ $\nu = 0.15$
Connections:	
Pure Rocking	$E = 12.6 \text{ GPa}$ $\nu = 0.15$
Rocking w/slip	$E = 41.3 \text{ GPa/per meter of connection thickness}$ $G_d = 18.0 \text{ GPa/per meter of connection thickness}$ $G_s = 0.18 \text{ GPa/per meter of connection thickness}$
Post-tensioning (6 bars):	
Area (1 bar) = 500 mm^2	
Initial Steel Stress = 687 MPa	
Gravity Loads:	
Roof Level = 507.1 kN	
Floor Level = 670.4 kN	
Masses:	
Roof Level = 51700 kg	
Floor Level = 68300 kg	

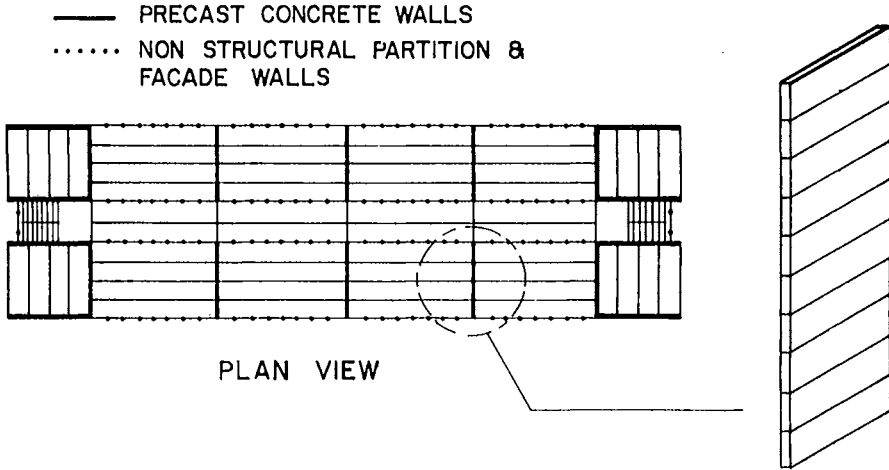


Fig. 1--Typical American cross-wall configuration

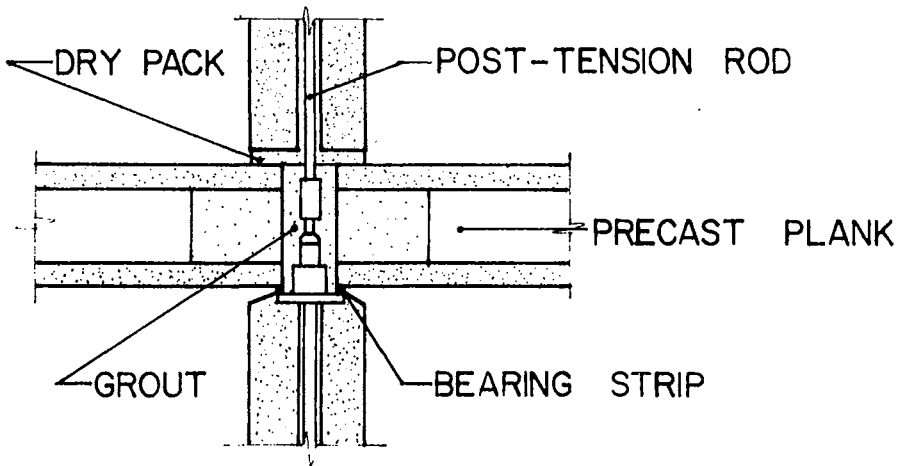


Fig. 2--Horizontal connection-section

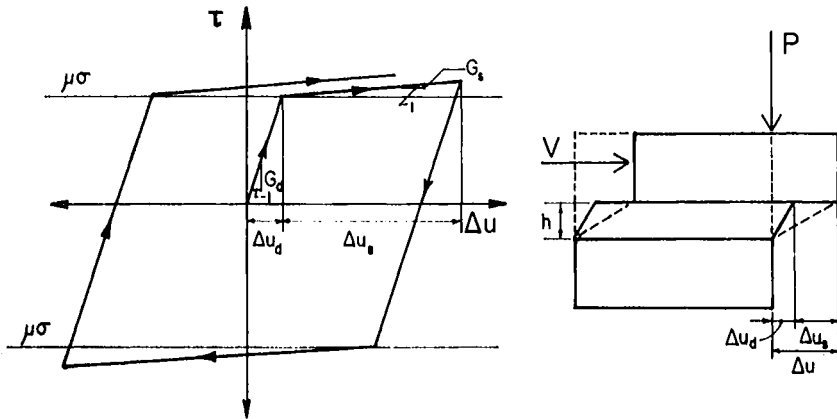


Fig. 3--Shear material model for horizontal connection

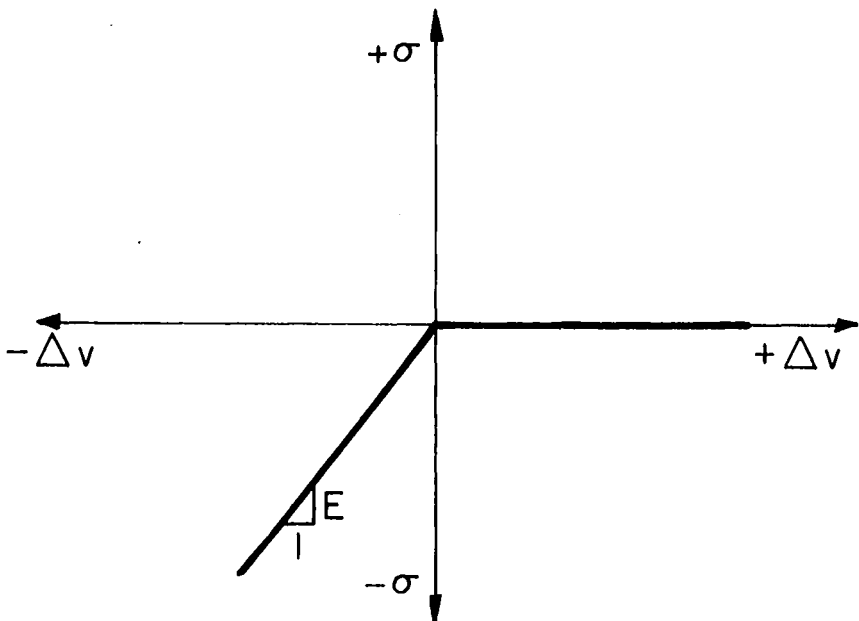


Fig. 4--Axial material model for horizontal connection

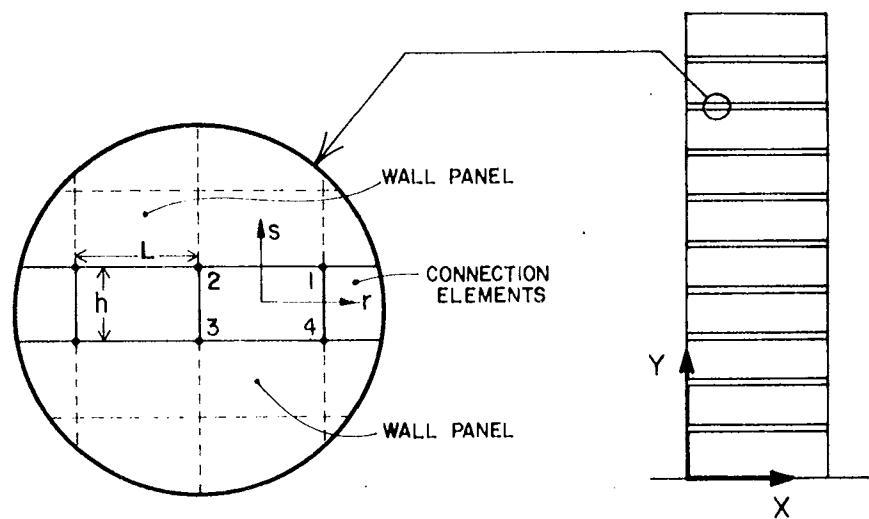


Fig. 5--Finite element discretization of connection region

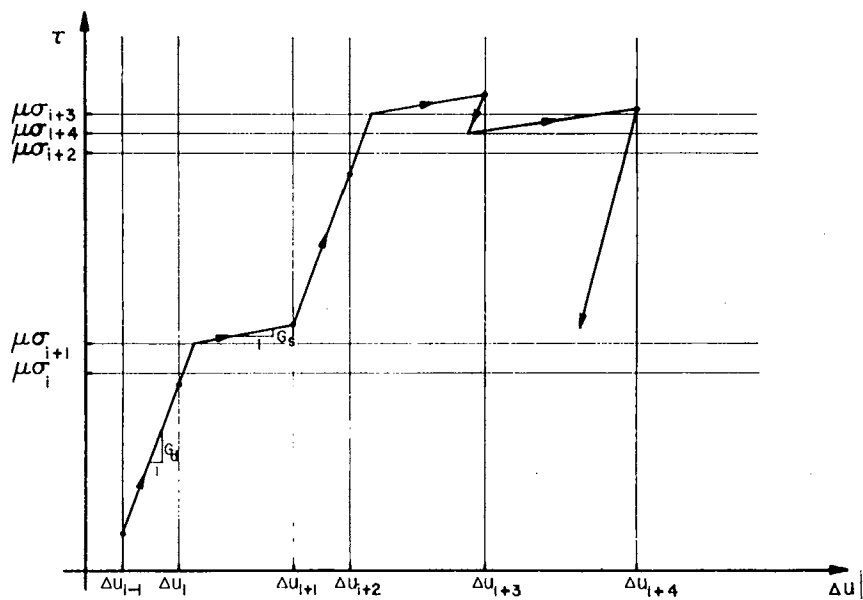


Fig. 6--Possible shear stress-shear deformation behavior

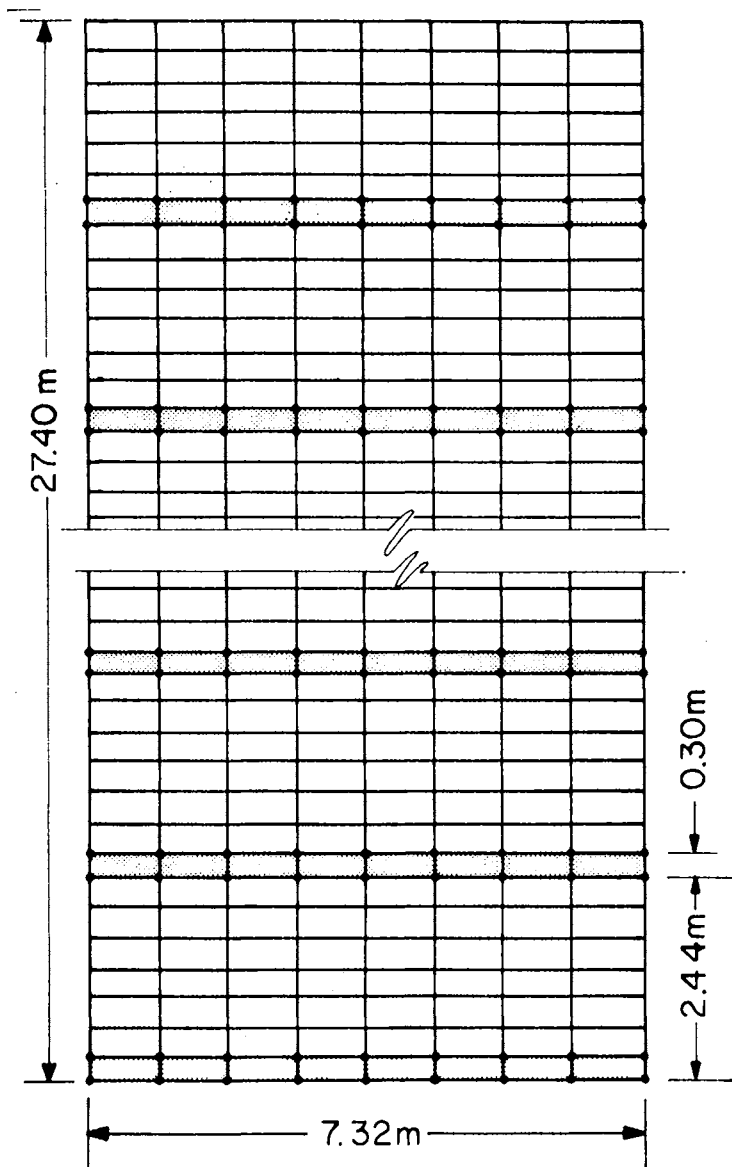


Fig. 7--Discretizations of isolated cross-wall

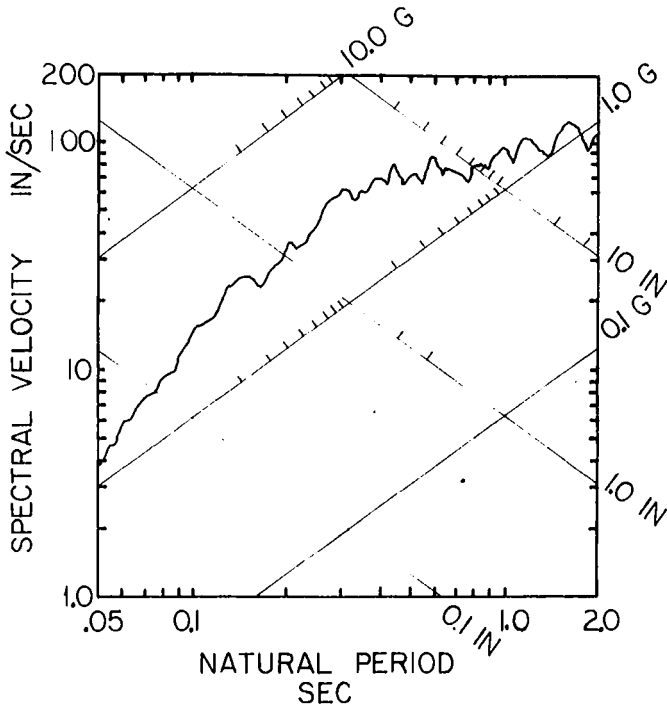


Fig. 8--Response spectra peak acceleration 1.0g

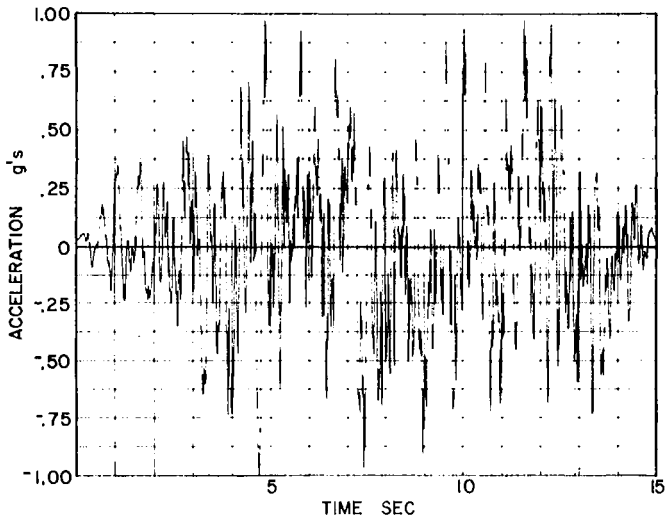


Fig. 9--Acceleration time history

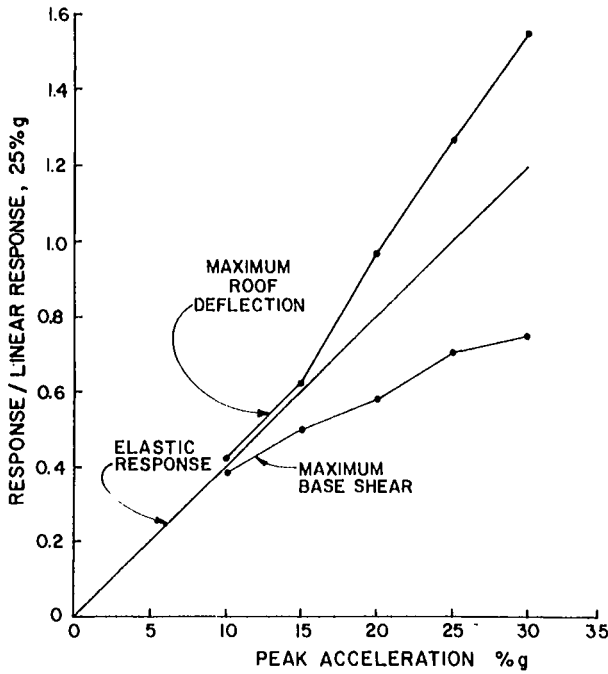


Fig. 10--Effect of rocking

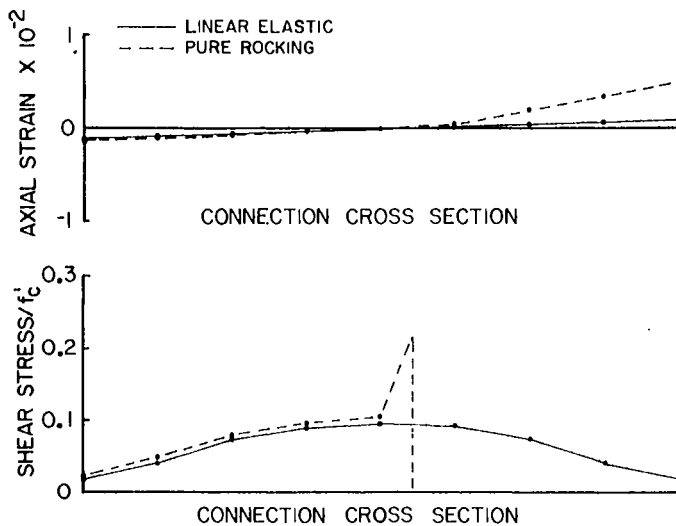


Fig. 11--Axial strain and shear stress distribution along second level horizontal connection--Rocking, 15%g

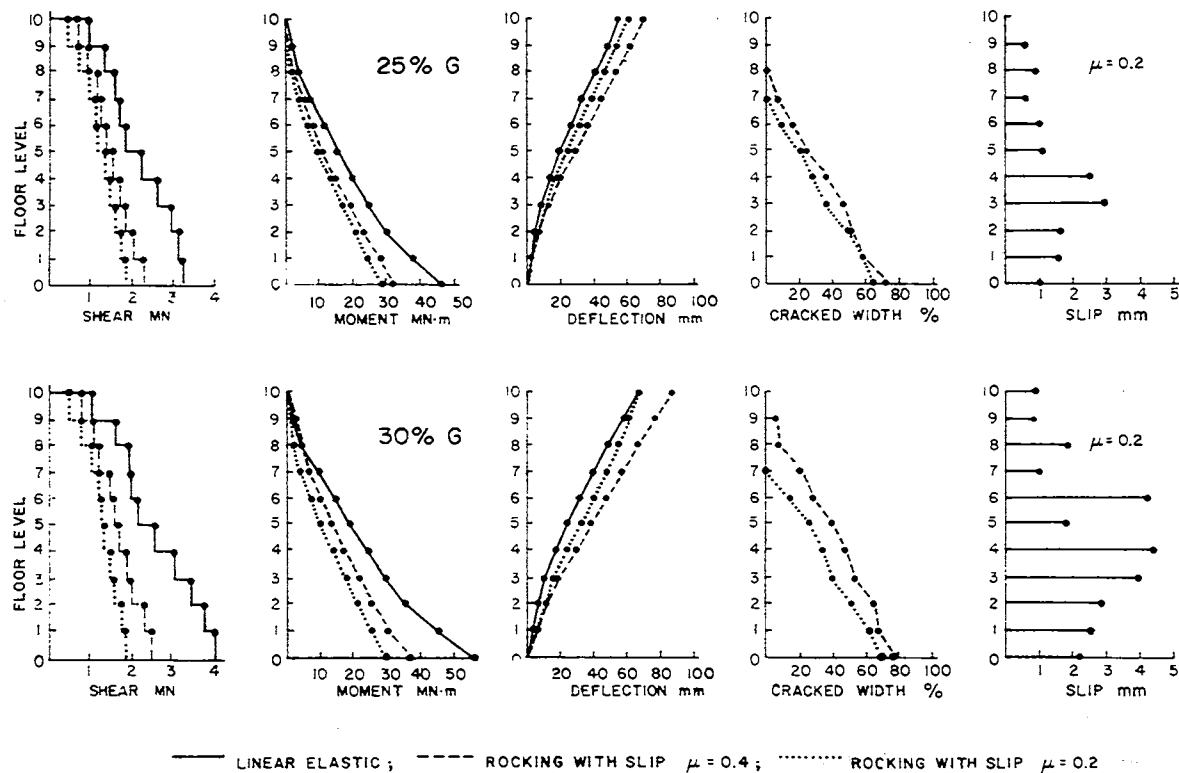


Fig. 12--Effect of rocking with shear slip

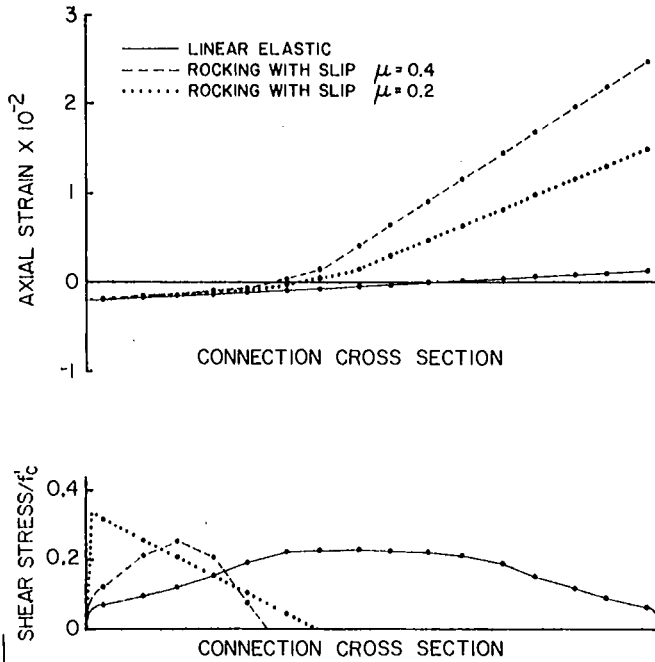


Fig. 13--Axial strain and shear stress distributions along second level horizontal connection--Rocking with slip, 30%g

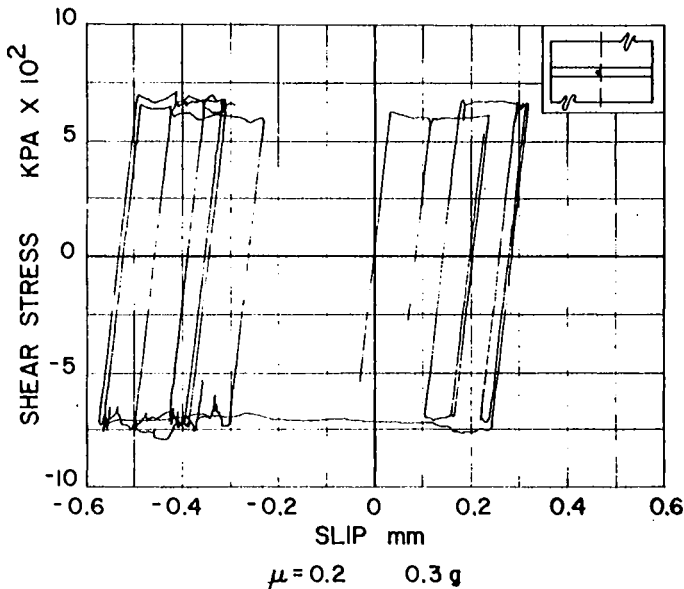


Fig. 14--Hysteretic behavior at a point near the neutral axis

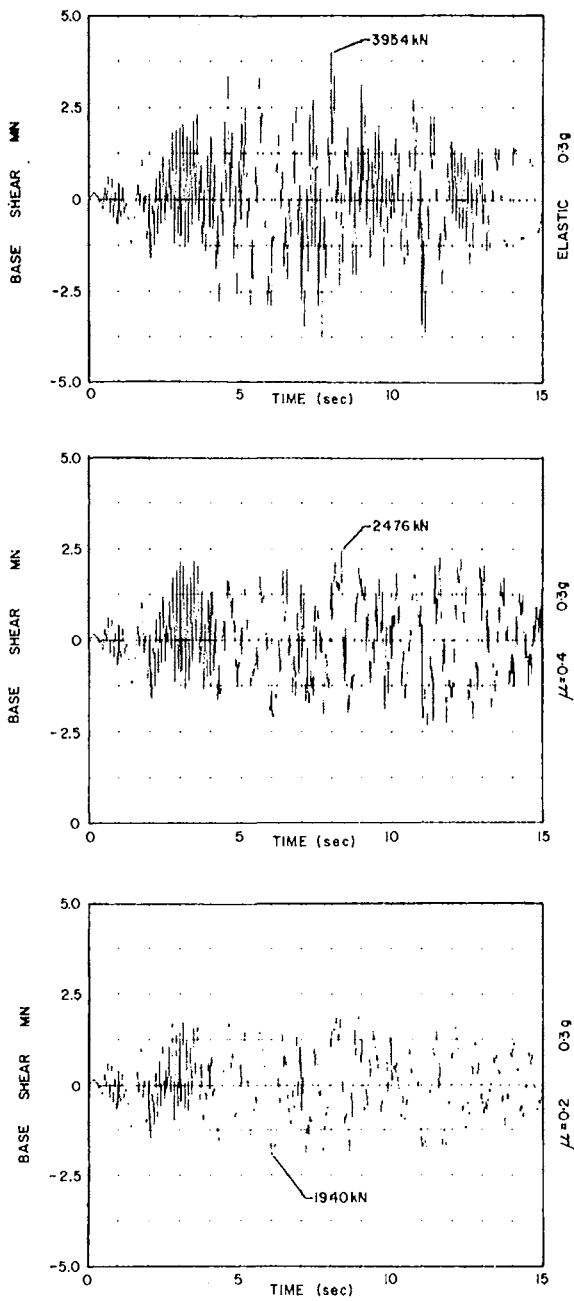


Fig. 15--Comparison of base shear time histories

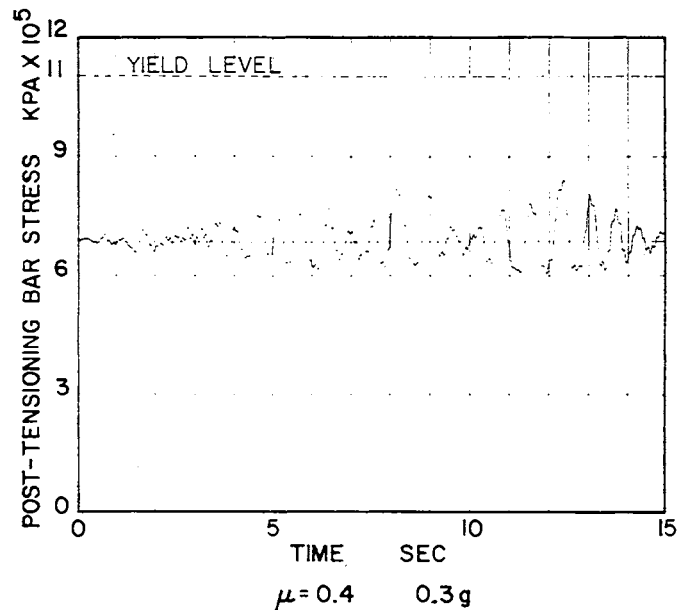


Fig. 16--Response of typical post-tensioning bar

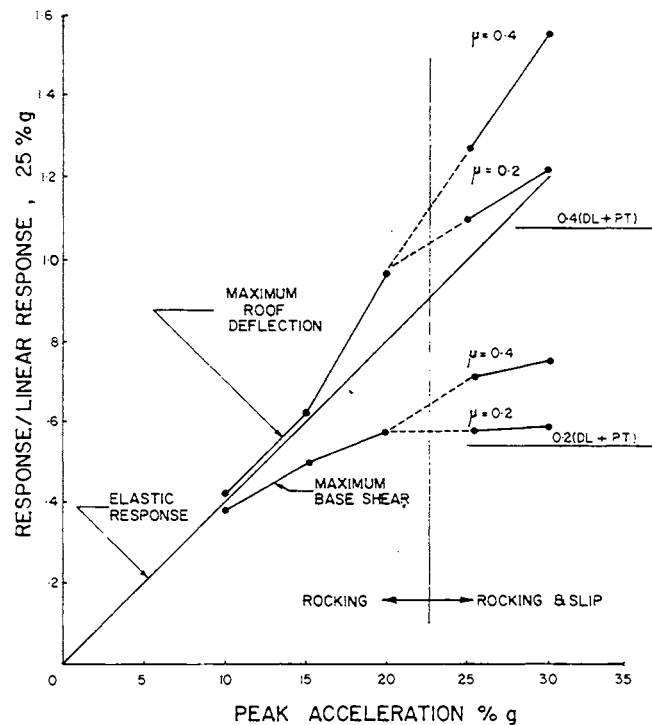


Fig. 17--Effect of nonlinear-inelastic connection behavior

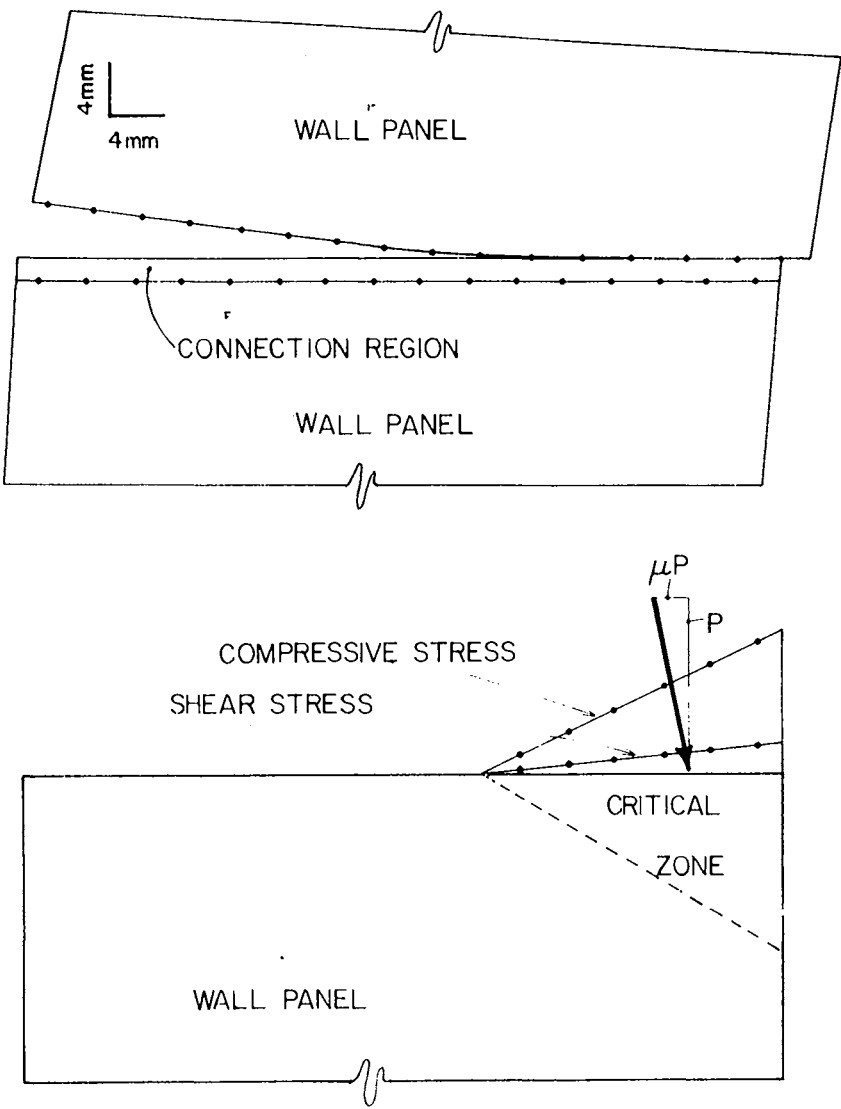


Fig. 18--Opening, slippage, and resulting stress distributions along horizontal connection

Finite Element Predictions of the Behavior of Slab-Column Connections Transferring Moment

By Jun Yamazaki and Neil M. Hawkins

Synopsis: Results are reported of the correlation between finite element predictions of the strength and behavior of flat slab to interior column connections transferring moment and response observed in tests on such connections. Predictions of the elastic response of the connections were made using a general elasticity program and a plate bending program. Although the general elasticity program considered shear deformations, it did not provide a markedly better correlation with the test data than the plate bending program. The plate bending analysis was extended into the inelastic range using an incremental procedure that recognized variations in stiffness and yielding of the slab with the directions of the reinforcement and principal moments. While the measured strains in the reinforcement and concrete were in reasonable agreement with the predictions, the measured deflections were about twice the predicted deflections. That discrepancy was found to be caused by bond slip of the reinforcement passing through the column and the inability of the program to correctly assess the torsional cracked section stiffness of reinforced concrete.

Keywords: building codes; columns (supports); concrete slabs; connections; elastic analysis; finite element method; flat concrete slabs; lateral pressure; mathematical models; moments; reinforced concrete; shear properties; shear strength; stiffness; structural analysis.

Jun Yamazaki is Associate Professor of Civil Engineering, Tokyo Metropolitan University, Tokyo, Japan.

Neil M. Hawkins is Professor and Chairman, Department of Civil Engineering, University of Washington, Seattle, Washington.

1. INTRODUCTION

When flat plate structures resist lateral loads, moments are transferred between slab and column. Where flat plate framing is used as the gravity load carrying system only, moment transfer still occurs at edge, corner, and first interior columns. Further, for earthquake loadings it is necessary to check that the vertical load carrying capacity of system is still adequate at the moments induced by the maximum likely distortions (1). Flat slab structures are inherently ductile. Tests (2) have shown that under gravity loads extensive redistributions of moments can be achieved in the slab prior to collapse provided a punching failure can be avoided at the slab to column connection. The punching capacity of such connections decreases as the moment transferred to the column through the connection increases. The amount of moment transferred depends on the stiffness of the connection. In several tests on slab systems (2) failures have initiated at connections at which moment transfer was observed or was possible and was not taken into account in the design. The lack of understanding of the factors controlling the strength, stiffness and ductility of slab to column connections constrains the use of flat plate structures, especially as part of the main lateral load resisting system.

Kinnunen and Nylander (3, 4) studied the behavior of axisymmetric slab to column connections transferring shears and constructed an idealized physical model capable of predicting the behavior they observed in tests. Although inclined cracks developed in their specimens at as little as 50 percent of the punching capacity, those cracks had little effect on the stiffness of their connections and did not lead to immediate failure. As predicted by their model, the strength, stiffness and ductility of the connections depended primarily on the circumferential stiffness of the slabs. That stiffness could be increased by increasing the amount or changing the distribution of the slab reinforcement. By studying the simplest possible loading case, Kinnunen and Nylander demonstrated that to correctly predict the behavior of slab to column connections, analyses must take proper account of the inelastic effects that develop before failure. Obviously, considerable difficulties can be expected in developing a physical model to predict the behavior of slab to column connections transferring moment.

Over the last decade, the use of finite element methods of analysis for complex engineering structures has become routine. However, the limitations to that use, especially for reinforced concrete structures, are not well understood. This paper contributes to that understanding by examining the correlation between finite element predictions of the strength and behavior of flat slab to column connections transferring moments and the response observed in tests on such connections.

2. TEST SPECIMENS

Tests have been conducted at the University of Washington on ten specimens (5, 6) having the properties listed in Table 1 and the proportions shown in Fig. 1. Six of those specimens contained integral beam shear reinforcement made up of closed hoops surrounding the four top and bottom reinforcing bars from the slab that passed through each column face. The size and spacing for that shear reinforcement and the distance that reinforcement extended out into the slab from each column face are shown in Table. 1. Half of the specimens from each group were subjected to a low moment loading and half to a high moment loading. The proportions and locations for the forces applied to the specimens to produce those loadings are shown in Fig. 1. For each specimen, the first three numerals indicate the flexural reinforcement ratio for the upper mat of the slab; the characters A and E indicate specimens without and with shear reinforcement respectively; the characters L and H indicate a low or high moment loading; and a final set of three numerals indicates the percentage of shear reinforcement in a 12-inch (305 mm) wide section framing into the column. The reinforcement patterns for the top and bottom mats of the slab are shown in Fig. 2 along with the locations for the electrical resistance gages placed on those mats. Exactly 0.75-inch (19 mm) cover was provided over the outer bars of each mat. The loads indicated in Fig. 1 were held in constant proportion to one another and increased monotonically until failure occurred after twelve or more load increments. Comprehensive details of the test specimens and results are provided in References 5 and 6.

3. FINITE ELEMENT MODEL

The prime finite element program used for this study was the plate bending program developed by Clough and Felippa (7). Six basic assumptions were used in developing the finite element model for the test specimens. Those assumptions concerned: 1) mesh size; 2) fixity of the connection between the slab and the column; 3) shear distortions; 4) isotropy; 5) stiffness; and 6) behavior in the inelastic range.

Mesh Size. The mesh layout used in this study is shown in Fig. 3. The smallest element was located adjacent to the column and was 3-inches (76 mm) square. The side length for that element was half the slab thickness for the specimen, one-quarter of the side length for the column and approximately 0.4 times the spacing of the flexural reinforcement in the slab. Cracking of the concrete