Behavior of Large Panel Precast Coupled Wall Systems Subjected to Earthquake Loading

Andrew Scanlon
Professor of Civil Engineering
The Pennsylvania State University
University Park, Pennsylvania

M. Reza Kianoush
Structural Design Engineer
Gore and Storrie Ltd.
Toronto, Canada

Before large panel concrete wall structures can be used with confidence in seismic zones, a clear understanding of their behavior under potentially severe earthquake loading is required. Such understanding can be provided by a combination of experimental investigation in the laboratory, analytical studies, and observation of performance of structures that have been constructed in various parts of the world.

Fintel[1] reported that precast panel buildings performed well during the 1977 Romanian earthquake. However, large panel precast wall systems have not yet found general acceptance for resisting earthquake forces in North America. At a recent PCI sponsored workshop on the Effective Use of Precast Concrete for Seismic Resistance,[2] the need for research in this area was emphasized.

Several experimental investigations on behavior of large panel wall systems have been reported in recent years.

Oliva and Shahrooz[3] conducted shaking table tests of one-third size scale models of three story systems consisting of solid walls, walls with door openings, and wall panels with adjoining and flange walls. Shear slip motion was constrained by shear keys. The result was that limited shear slip took place and behavior was dominated by rocking. Stress concentrations were induced at wall ends, leading to concrete crushing as a result of rocking motion.

Velkov et al.[4] tested specimens similar to those tested by Oliva and Shahrooz. Quasi-static loading was used and displacement ductility ratios in excess of five were reported. It was concluded that large panel systems with carefully designed connection details can be used in seismic zones.

Harris and Caccese[5] tested 1/32 scale models of simple walls, five stories in height, using a shaking table. The experimental program was undertaken to investigate behavior of simple walls studied analytically by Becker and Llorente.[6] Behavior of the scale models was similar to the behavior predicted analytically. The failure mechanisms were shown to be slip between panels and crushing of panel corners caused by rocking.

In addition to the analytical studies of isolated walls reported by Becker and Llorente,[6] a similar investigation with somewhat different modeling techniques was undertaken by Schricker and Powell.[7] Both of these studies highlighted the influence of slip between panels and rocking of the panels on the behavior of isolated wall systems.

Chu et al.[8] tested a one-fifth scale model of an eight story coupled wall building constructed in Beijing, China. Both static and dynamic loads were applied. Dynamic loading was applied with an exciter placed at roof level. The results indicate that large panel coupled systems possess adequate energy dissipation characteristics and that, even if the walls are damaged, the structural stability/integrity can be maintained.

Paulay and his co-workers,[9,10] as well as other researchers,[11,12] have illustrated the effectiveness of coupling beams in improving the performance of cast-in-place structural walls under earthquake loading. The coupling beams provide a means of dissipating energy during earthquake motion without compromising the gravity loadbearing capacity of the walls. The present study was conducted to investigate the potential for coupling beams to perform a similar function in large panel precast wall systems.

Synopsis

Seismic response of large panel wall systems connected by coupling beams is investigated using dynamic inelastic computer modeling techniques. Effects of coupling beam strength and stiffness are studied as well as the method of providing vertical continuity. Coupling beams are shown to improve behavior significantly compared to isolated walls. Design and construction considerations are discussed.

ANALYTICAL MODELING

The analytical modeling techniques used in the study are described in detail in Refs. 13 and 16 (see also Appendix A). The following models were implemented in the general purpose computer program, DRAIN-2D, developed by Kanaan and Powel.[14]

(a) **Wall panels** — Plane stress elements to model linear elastic behavior of concrete panels with horizontal and vertical bars.

(b) **Horizontal joints** — No tension across connections to allow for gap opening, nonlinear behavior of concrete in compression, elasto-plastic behavior of mild steel reinforcement across the joint, linear elastic behavior of post-tensioning bars, shear-slip mechanism between panels without mild reinforcement, and shear-friction mechanism between panels connected by mild reinforcement.

(c) **Coupling beams**

— Slender beams with inelastic action modeled by plastic hinges at the beam ends.

— Deep beams reinforced with diagonal bars modeled by inelas-

Table 1. Maximum internal forces based on linear elastic frame analysis.

Coupling beam type	Maximum base moment kN•m (k•ft)	Maximum base shear kN (kips)	Maximum base axial force kN (kips)	Maximum beam moment kN•m (k•ft)	Maximum beam diagonal force kN (kips)
Slender beams	10006 (7374)	1051 (236)	10546 (2371)	413 (305)	— —
Deep beams	13460 (9920)	1016 (228)	9691 (2179)	— —	278 (63)

* Deep beam modeled as diagonal truss elements.

tic truss elements aligned with the diagonal bars.

The horizontal joint models are based on the work of Becker and Llorente.[6] The modeling of slender coupling beams is similar to the procedure used by Saatcioglu[11] for cast-in-place coupled walls. Using truss elements to model deep beams reinforced with diagonal bars underestimates the stiffness at early stages of loading but is considered to be a reasonable representation of behavior at high load intensities when diagonal cracking and spalling occur as indicated in laboratory tests.[10]

To determine an appropriate range of yield strength parameters for the coupling beams, an equivalent static load analysis of the structure was made following the procedure outlined in the National Building Code of Canada[15] in which a total base shear is calculated based on the weight of the structure, geographical location (seismic zone) and several other factors. A similar design approach is used in other North American design codes.

A set of lateral forces calculated from the design base shear is applied over the height of the structure. Internal forces are calculated based on a linear elastic analysis of the structure subjected to these forces. Results of this analysis are summarized in Table 1. Further details of the analysis are given in Ref. 16. Based on the results of the analysis, appropriate ranges of yield moment (slender beams) and yield force in diagonal bars (deep beams) were selected for study.

Mild steel vertical reinforcement provided across the connections consisted of 0.25 percent of the gross area, uniformly distributed, plus 0.5 percent concentrated near the two edges of the panels. This reinforcement satisfies minimun requirements of Section A of the 1977 ACI Code[17] dealing with seismic design of special shear walls, and was also adequate to resist the base shear, moment, and axial force calculated from the static analysis.

In cases where post-tensioned bars were provided for vertical continuity, five post-tensioning bars were placed uniformly in each wall to provide a total force equal to the yield capacity of the mild steel reinforcement.

The range of initial stiffness values for coupling beams was also established on the basis of the values used in the equivalent static analysis.

In total, fifteen structures with properties listed in Tables 2 and 3 were considered in the study.

The assumption that the wall panels remain linear elastic ignores effects of localized nonlinear behavior adjacent to coupling beams and in the vicinity of high stress regions caused by rocking. However, since the majority of the panel is expected to remain in the linear elastic range, it is assumed that the overall behavior is adequately modeled. A limited evaluation of the analytical modeling techniques is described in Ref. 13. Limitations of the analytical model described above should be recognized in interpreting the results of the parametric study described in the following section.

RANGE OF PARAMETERS CONSIDERED

A coupled wall arrangement forming part of the lateral load resisting system of a ten story apartment building was selected as a basis for the parametric study. The general configuration, including overall dimensions, is shown in Fig. 1. Horizontal connections were assumed to be of the platform type as shown in Fig. 2.

Slender coupling beams and deep coupling beams were considered in the study. Slender beams were considered to have top and bottom reinforcement such that a plastic hinge could be developed at each end of the beam. It was assumed that adequate shear strength would be provided to permit full development of the plastic hinges. Deep beams were assumed to be reinforced with diagonal bars as proposed by Paulay and Binney[10] for cast-in-place coupled walls and that detailing of the reinforcement was adequate to permit yielding of the diagonal bars without premature failure under high intensity cyclic load.

The main parameters considered were the yield strength of the coupling beams, initial stiffness of the coupling beams (prior to yielding), and the method of providing continuity across the horizontal joints, i.e., mild reinforcement or post-tensioning.

SELECTION OF EARTHQUAKE RECORD

Before proceeding with the parametric study, it was necessary to select an appropriate ground motion acceleration-time history for use as input to the analysis. The following three strong ground motion acceleration-time histories were considered: (1) El Centro, 1940, N-S Component, (2) Taft, July 1952, and (3) Pacoima Dam, 1971, S16E Component. The acceleration-time history records were obtained from the Earthquake Engineering Research Institute.[18]

To provide a basis for comparison, the ordinates of the acceleration-time histories were adjusted to yield spectrum intensities equal to 1.5 times the spectrum intensity of the El Centro record. The spectrum intensity is defined as the area under the 5 percent damped relative velocity response spectrum between periods of 0.1 and 3 seconds. This is the same approach used by Saatcioglu[11] to obtain a high intensity ground motion.

A duration of 10 seconds was used in this study. Integration time steps of 0.01 and 0.001 seconds were used for elastic and inelastic analyses, respectively.

The acceleration-time histories were applied to the structure illustrated in Fig. 1, using the following analytical models:

1. Wall panels — linear elastic
2. Connections
   — zero-tension axial model
   — shear slip model
3. Post-tensioning bars — linear elastic
4. Slender coupling beams — linear elastic

Two sets of analyses were completed. In the first, the structure (including connections) was assumed to remain linear elastic throughout. For this case the Pacoima Dam record produced the most severe response. In the second set, nonlinear behavior was permitted in the

Table 2. Coupling beam parameters and vertical continuity for structures analyzed.

Structure No.	Coupling beam parameters						Vertical continuity
	Slender beams*		Deep beams†				
	M_y (kN•m)	EI (kN•m²)	L/h	A_s (mm²)	f_y (MPa)	F_y (kN)	
1	(Simple wall — no coupling beams)						Reinforced
2	(Simple wall — no coupling beams)						Post-tensioned
3	90	122 060	—	—	—	—	Reinforced
4	180	122 060	—	—	—	—	Reinforced
5	Elastic	122 060	—	—	—	—	Reinforced
6	90	122 060	—	—	—	—	Post-tensioned
7	180	122 060	—	—	—	—	Post-tensioned
8	270	122 060	—	—	—	—	Post-tensioned
9	180	122 060	—	—	—	—	Reinforced plus post-tensioned
10	180	61 030	—	—	—	—	Post-tensioned
11	180	183 090	—	—	—	—	Post-tensioned
12	—	—	1.0	600	300	180	Reinforced
13	—	—	1.0	600	300	180	Post-tensioned
14	—	—	1.4	600	300	180	Post-tensioned
15	—	—	1.8	600	300	180	Post-tensioned

* Strain hardening stiffness after yielding = 5 percent of initial elastic.

† Strain hardening stiffness after yielding = 8 percent of initial elastic.

Conversion factors: 1 mm = 0.0394 in.; 1 m = 3.28 ft; 1 kN = 0.225 kips; 1 MPa = 145 psi.

Table 3. Details of wall panels and horizontal connections.

Wall panels	Horizontal connections	
	Reinforced	Post-tensioned
E = 27 400 MPa $\nu = 0.15$ f'_c = 30 MPa ρ_{horz} = 0.25 percent ρ_{vert} = 0.25 percent (uniform) ρ_{vert} = +0.50 percent (concentrated) Panel thickness = 200 mm	Reinforcing bars E_s = 200,000 MPa f_y = 300 MPa ρ = 0.25 percent (uniform) ρ = +0.50 percent (concentrated) Concrete E_c = 13,700 MPa G_d = 6850 MPa $G_s/G_d = 0.1$ $G_{deg}/G_d = 0.002$ f'_c = 15 MPa Coefficient of friction $\nu = 0.2$	Post-tensioning bars E_s = 200,000 MPa A_s = 5.3 mm^2 (per bar) No. of bars = 5 (per wall) Concrete E_c = 13,700 MPa G_d = 6850 MPa f'_c = 15 MPa Coefficient of friction $\nu = 0.2$

Conversion factors: 1 mm = 0.0394 in.; 1 MPa = 145 psi.

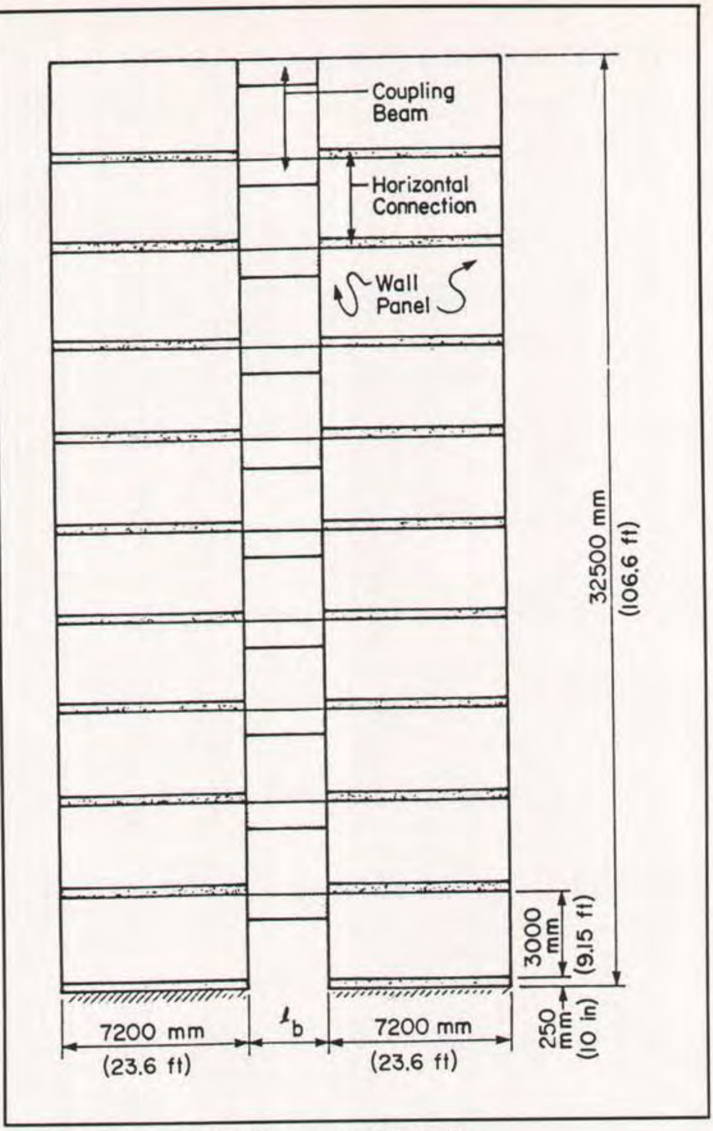


Fig. 1. Wall system used for parametric study.

connections. For this case Pacoima Dam produced the largest displacements and maximum slip. However, the largest base forces, beam moments and gap opening were produced by the Taft record. Results of the analyses are given in Figs. 3 and 4. The Taft record was then selected as the basis for the parametric study.

RESULTS OF PARAMETRIC STUDY

The fifteen structures listed in Table 1 were analyzed for the modified Taft earthquake record using an integration time step of 0.001 seconds. Detailed results of these analyses are given in Ref. 16. Presented in this section are se-

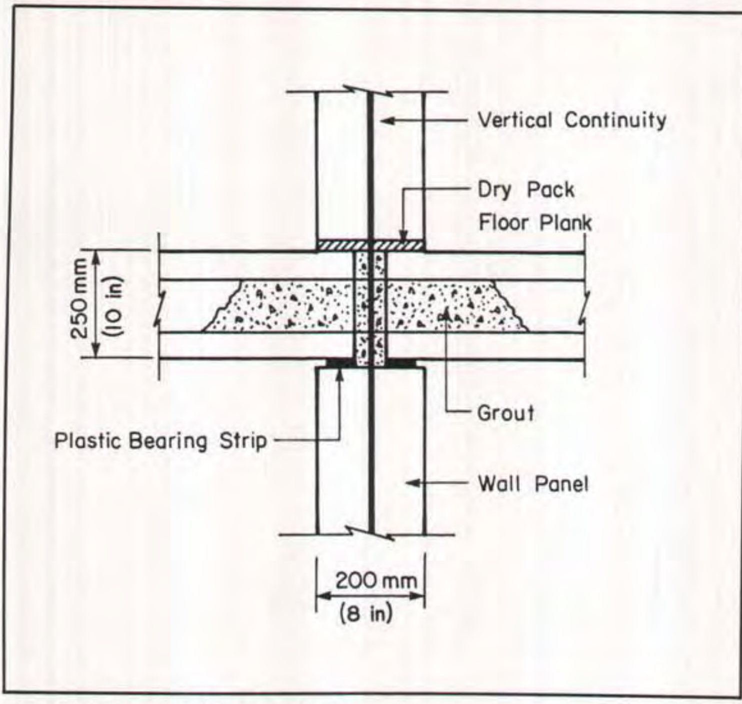


Fig. 2. Platform connection details.

lected results to illustrate the main trends in behavior. The results are given in terms of maximum response envelopes for various quantities as well as time histories for some of these quantities.

Effect of Variation in Yield Moment of Slender Beams

Results are presented first for structures with horizontal joints reinforced with mild steel bars for vertical continuity. Results are then given for structures with post-tensioning for vertical continuity.

Vertical continuity — reinforcing bars (Structures 1, 3, 4, 5) — Shown in Figs. 5 and 6 are the maximum horizontal displacement, maximum slip, maximum gap opening envelopes, and maximum strain distribution for M_y = 90 kN•m (66.4 k•ft). This structure did not show joint material failure. However, the coupling beam ductility demand was found to be excessive as shown in Fig. 7.

The beam ductility factor is the ratio of maximum end rotation to end rotation at first yield. Results are not presented for Structures 1, 4 and 5 of this series because joint material failure occurred prior to completion of the 10 seconds duration of input motion. Joint material failure was considered to occur at simultaneous steel yield in tension and concrete compressive strain in excess of strain at maximum stress.

Vertical continuity — post-tensioning (Structures 6, 7, 8) — Maximum response envelopes for this series are given in Figs. 8, 9 and 10. Comparing maximum slip and gap openings with those presented in Figs. 5 and 6 for the reinforced connection, it can be seen that the method used for providing vertical continuity has a significant effect on behavior. The reinforced connection shows considerably smaller slip and

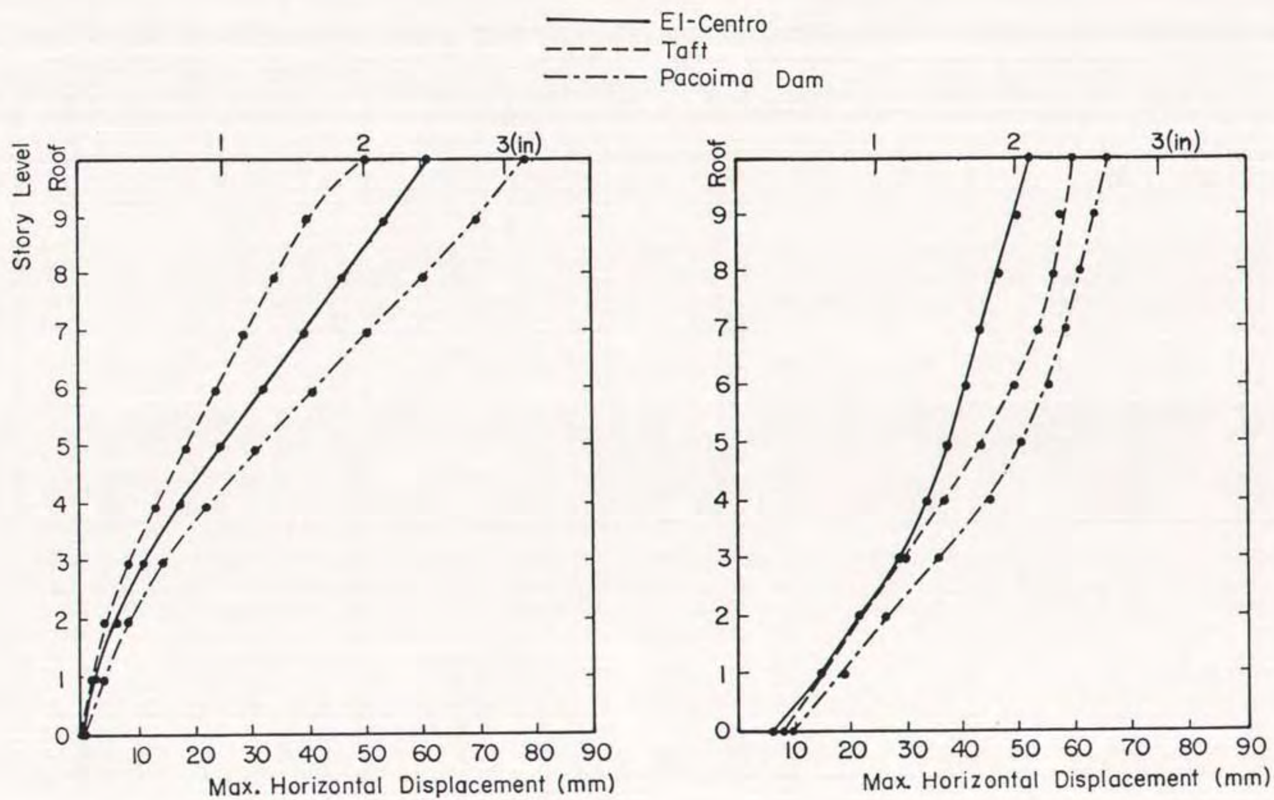


Fig. 3. Effect of different ground accelerograms on maximum displacements.

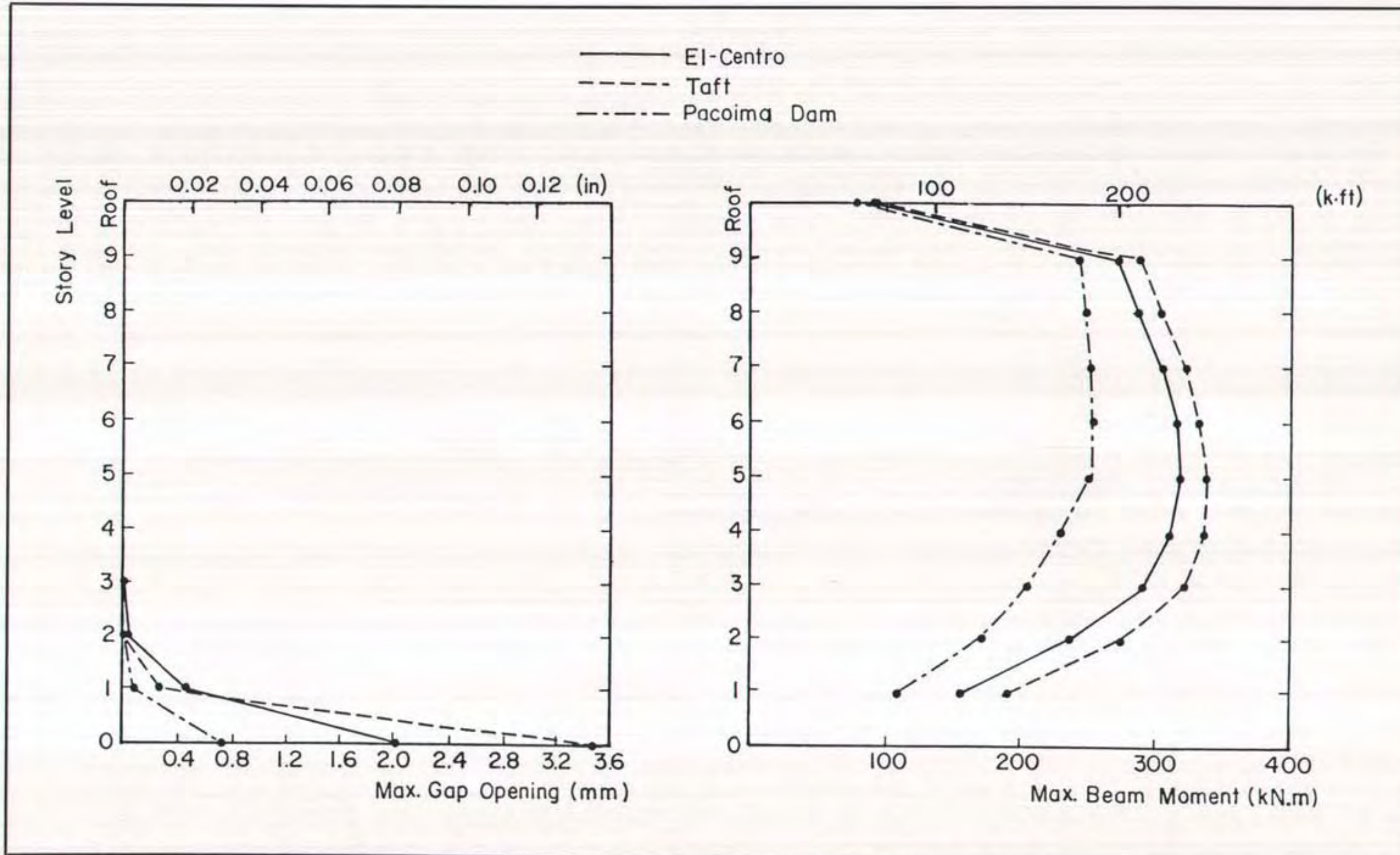


Fig. 4. Effect of different ground accelerograms on maximum gap opening and maximum beam moments.

larger gap opening than the post-tensioned wall.

The difference in behavior can be related directly to the properties of the assumed shear-friction (reinforced) and shear-slip (post-tensioned) relationships as shown in Figs. 11 and 12. In the shear slip model, the very low stiffness associated with the "post-yield" (i.e., G_s as shown in Figs. 11 and 12) portion of the force deformation relationship tends to cause isolation of the panels above the joint from the input motion when the friction force is overcome. The shear-friction mechanism, on the other hand, is considerably stiffer in the post-yield range and there is a greater tendency for over-turning, or gap opening, to dominate behavior.

Fig. 8 shows that the maximum slip envelopes are not significantly affected by variations in beam yield strength. However, Fig. 9 indicates that the maximum gap openings decrease significantly with increasing M_y. As indicated in Fig. 10, the ductility demands on coupling beams are significantly affected by the yield strength provided. In this case coupling beams with M_y = 90 kN•m (66.4 k•ft) show excessive ductility demands, particularly at higher stories, while for M_y = 180 kN•m (132.7 k•ft) and 270 kN•m (199.1 k•ft), the ductility demands are within practical ranges.

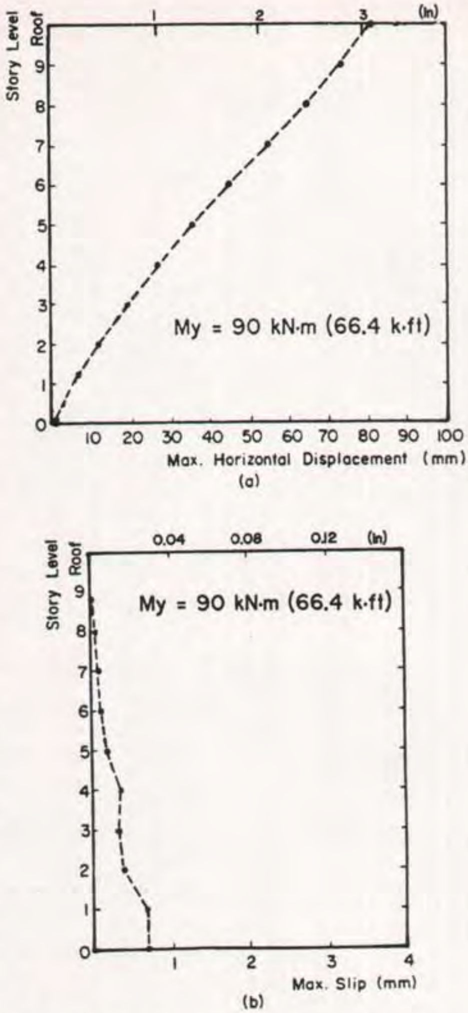


Fig. 5. Displacement and shear slip envelopes for M_y = 90 kN • m: slender beams, mild reinforcement for vertical continuity.

Effect of Variation in Initial Stiffness of Slender Beams

In this series (Structures 7, 10, 11), coupling beam yield strength was held constant at 180 kN•m (132.7 k•ft) and initial stiffness values (*EI*) were varied. Post-tensioning was used for vertical continuity. The results are presented along with results for a simple wall (no coupling beams) to provide a comparison between simple and coupled wall behavior.

Shown in Figs. 13, 14 and 15 are maximum envelopes for horizontal displacement, slip, gap opening, strain distribution, beam moment and beam ductility factor. In general, the maximum response quantities do not vary significantly with initial flexural stiffness of the coupling beams. This result is not surprising since the wall and beams are loaded well into the inelastic range and behavior is dominated largely

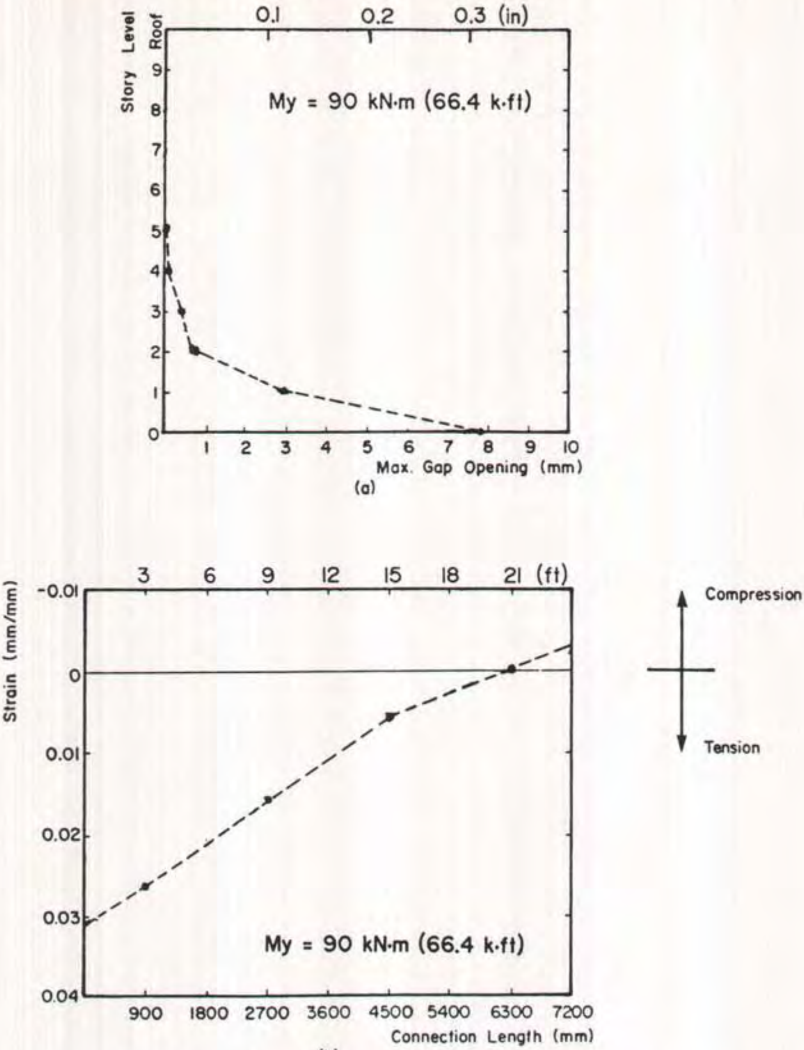


Fig. 6. Gap opening and strain distribution for $M_y = 90$ kN • m: slender beams, mild reinforcement for vertical continuity.

by the yielding of the coupling beams. It is worth noting, however, that the maximum *EI* value produced significantly higher ductility demands on the coupling beams than the other two cases.

A comparison between simple and coupled wall response quantities indicates important differences in behavior. In Figs. 13 and 14, the simple walls show significantly larger horizontal displacement and gap openings along horizontal joints, indicating that the coupling beams tend to resist overturning of the walls. Fig. 13 indicates smaller slip values for simple walls, particularly at lower stories.

The difference between behavior of

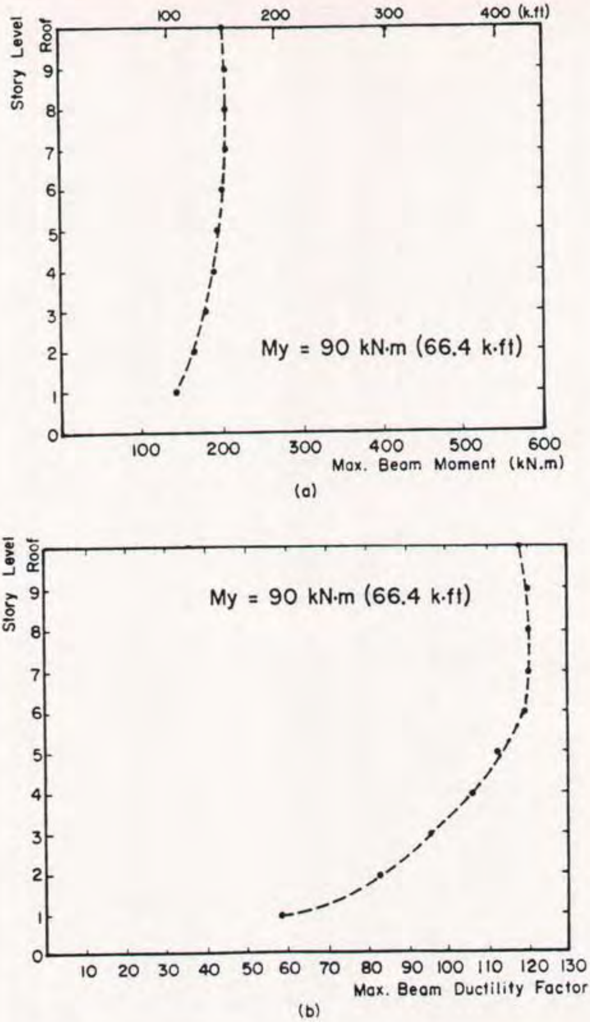


Fig. 7. Maximum beam moments and maximum beam ductility factor for $M_y = 90$ kN • m: slender beams, mild reinforcement for vertical continuity.

simple and coupled walls is further illustrated in Figs. 16 and 17 where the displacement-time and slip-time histories are presented. Up to about 3½ seconds, very little difference is evident between simple and coupled walls. However, beyond 3½ seconds the simple walls show considerably larger displacement amplitudes while the slip values for simple walls are generally smaller than for coupled walls. The difference in behavior was found to coincide with the onset of increased earthquake intensity.

Response of Walls with Deep Coupling Beams

A series of analyses was made on structures with deep coupling beams

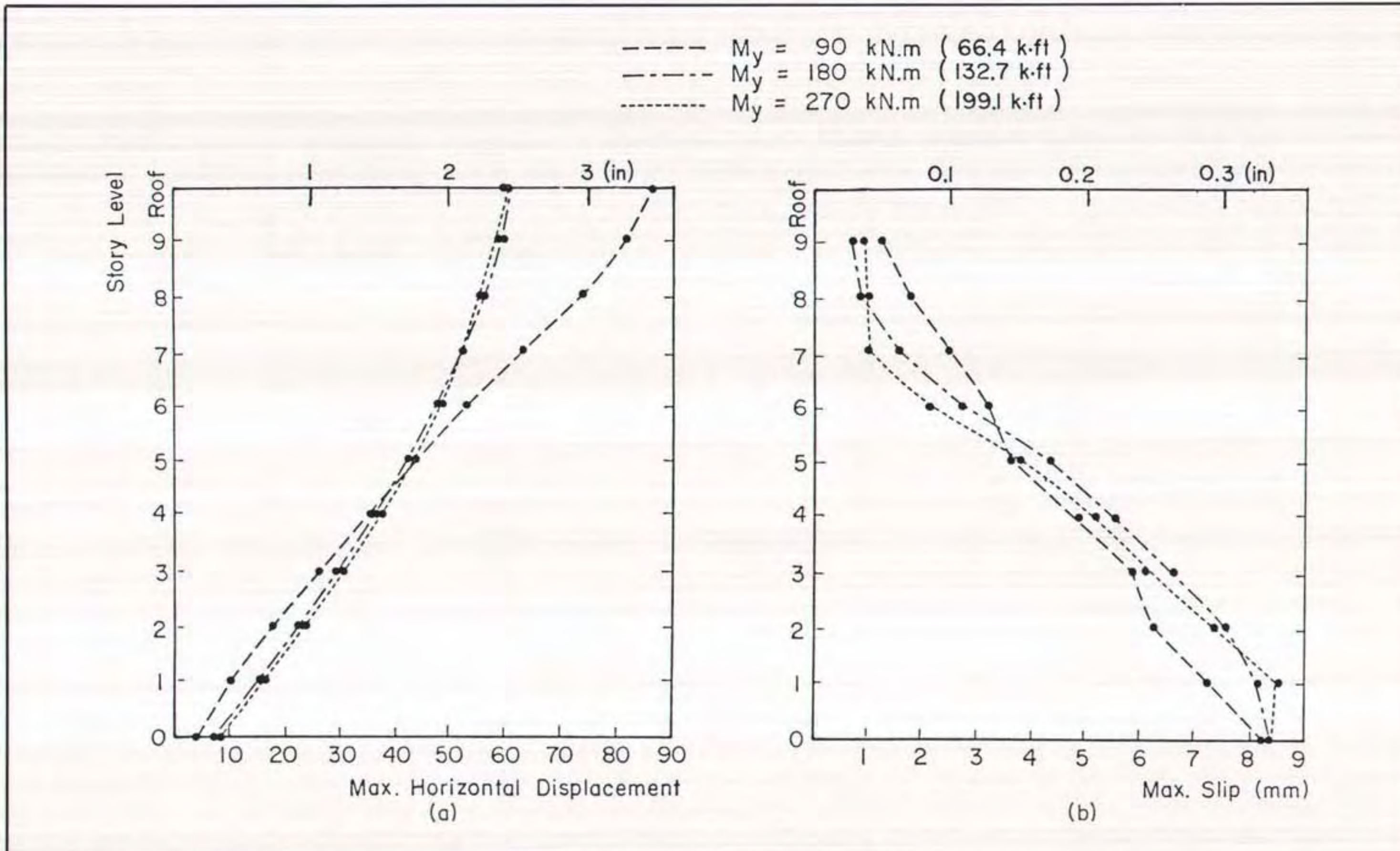


Fig. 8. Displacement and shear slip envelopes showing the effect of beam strength: slender beams, post-tensioning for vertical continuity.

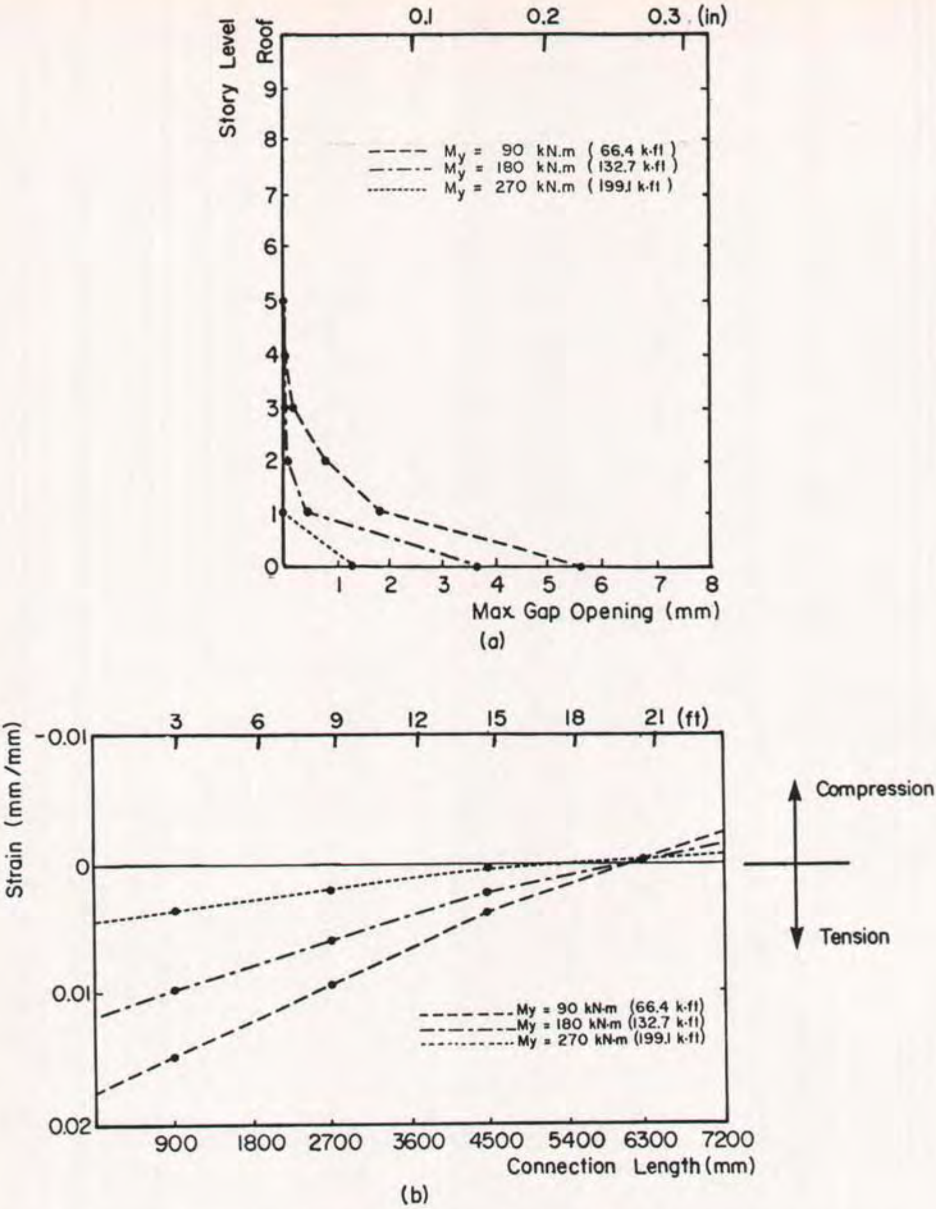


Fig. 9. Gap opening and strain distribution showing the effect of beam strength: slender beams, post-tensioning for vertical continuity.

reinforced with diagonal bars. Structure 12 with mild reinforcement across the horizontal joint suffered material failure at the horizontal joint before completing the 10 seconds of input motion. This is similar to the behavior observed in structures with slender beams which have mild reinforcement across horizontal joints.

Three additional analyses were made using post-tensioning for vertical continuity. In these analyses, the strength of

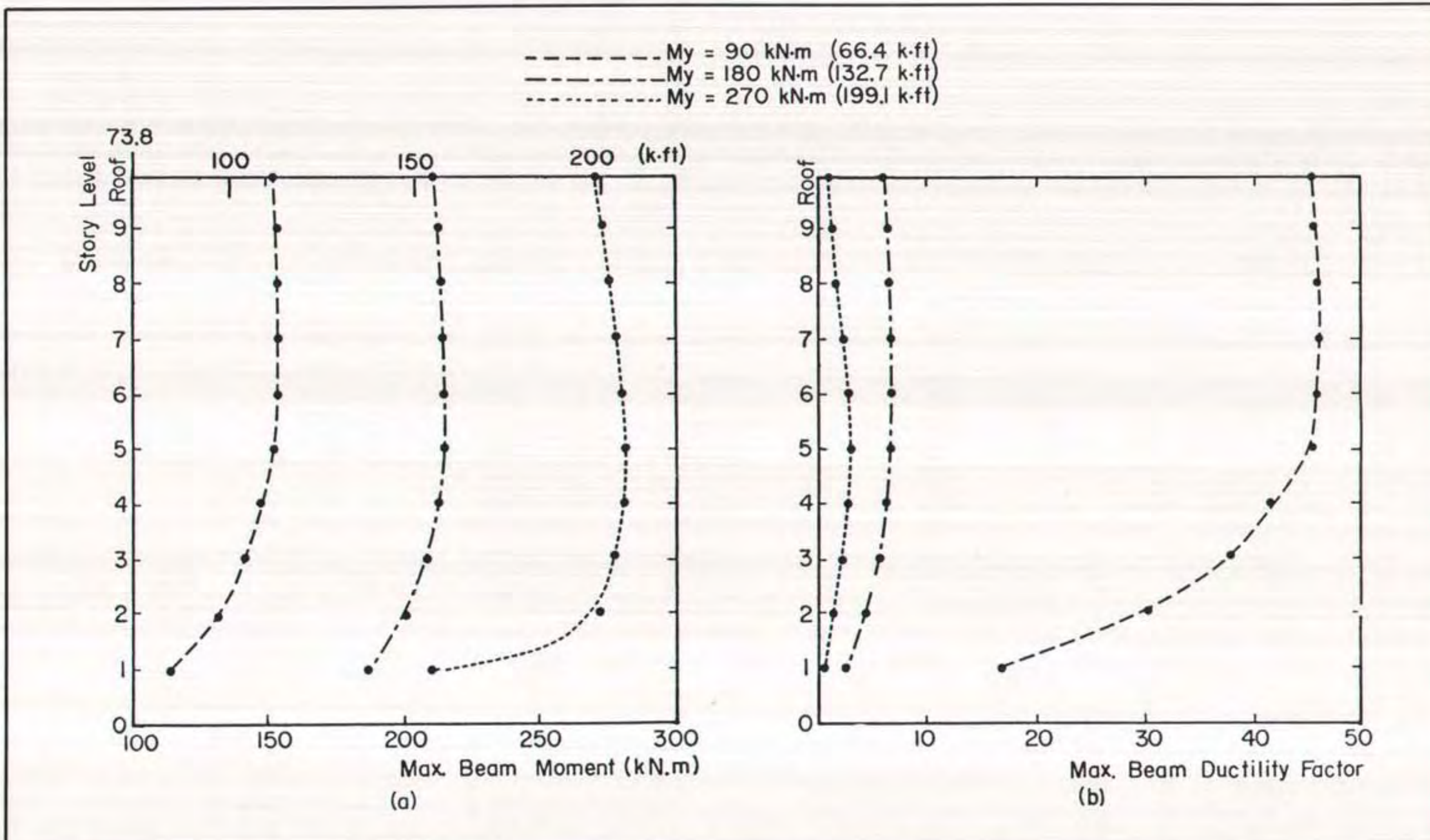
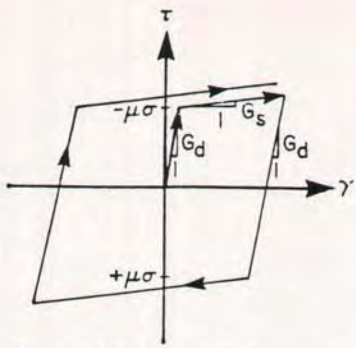
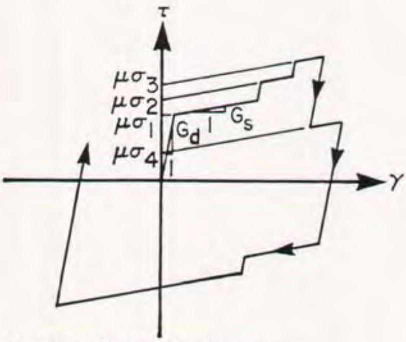


Fig. 10. Maximum beam moments and maximum beam ductility factor showing the effect of beam strength: slender beams, post-tensioning for vertical continuity.

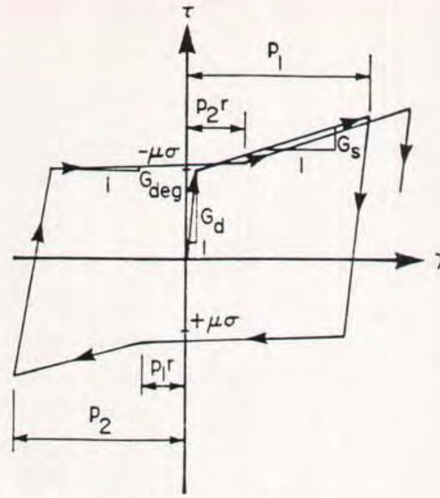


(a) Constant Normal Stresses

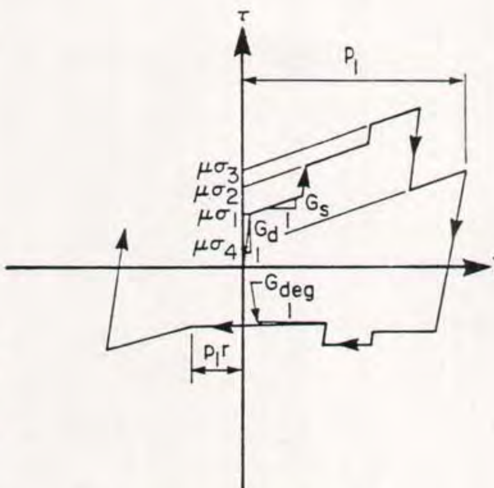


(b) Vary Normal Stresses

Fig. 11. Shear-slip model.



(a) Constant Normal Stresses



(b) Varying Normal Stresses

Fig. 12. Shear-friction model.

diagonal bars was held constant at a value of F_y = 180 kN (40.5 kips). The effective strength and stiffness, however, were varied through the beam span to depth ratio L/h, which was varied as indicated for Structures 13, 14 and 15 listed in Table 2.

Maximum response envelopes are presented in Figs. 18, 19 and 20. Also presented are results for a simple wall. Similar trends to those indicated for slender beams are clear in terms of the comparison between simple and coupled wall behavior. The coupling tends to reduce maximum displacement, gap openings and strain distribution along the connection while the coupled walls experience larger shear slip than the simple walls. Fig. 20 indicates that the bar ductility demands are generally less than a factor of four, which is considered to be within the practical range for axially loaded members. In modeling deep beams as diagonal truss members, the bar ductility factor is taken as the maximum bar axial strain divided by the elastic axial strain.

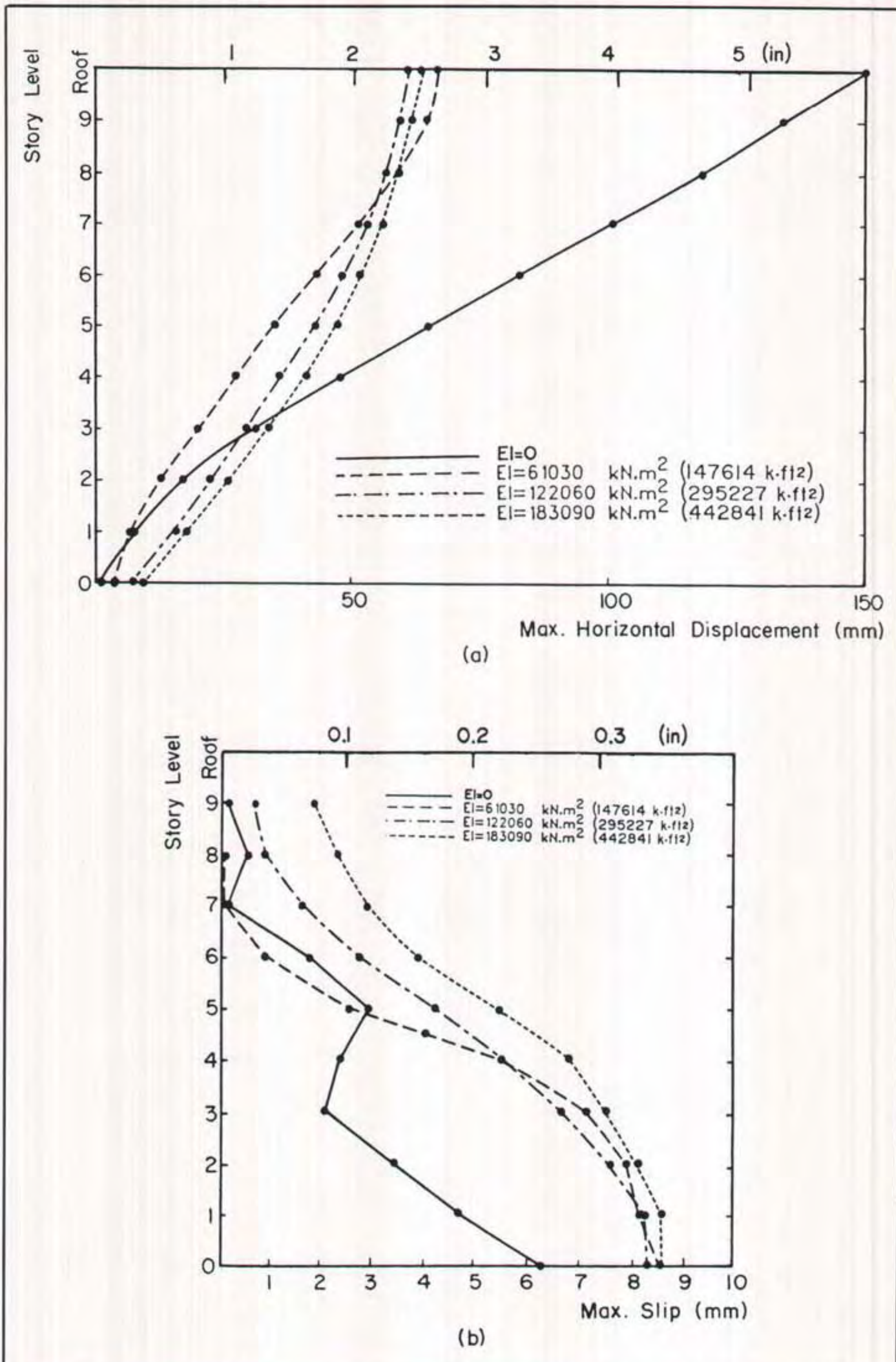


Fig. 13. Displacement and shear slip envelopes showing the effect of beam stiffness: slender beams, post-tensioning for vertical continuity.

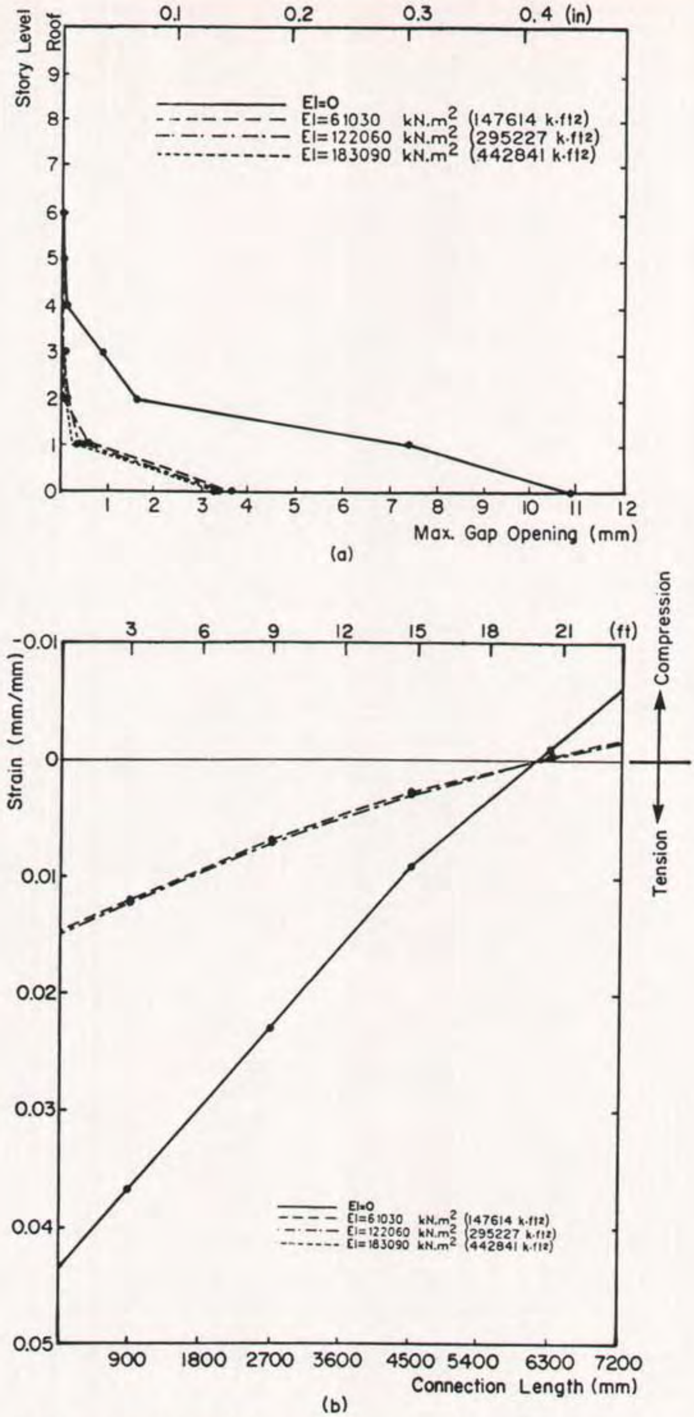


Fig. 14. Gap opening and strain distribution showing the effect of beam stiffness: slender beams, post-tensioning for vertical continuity.

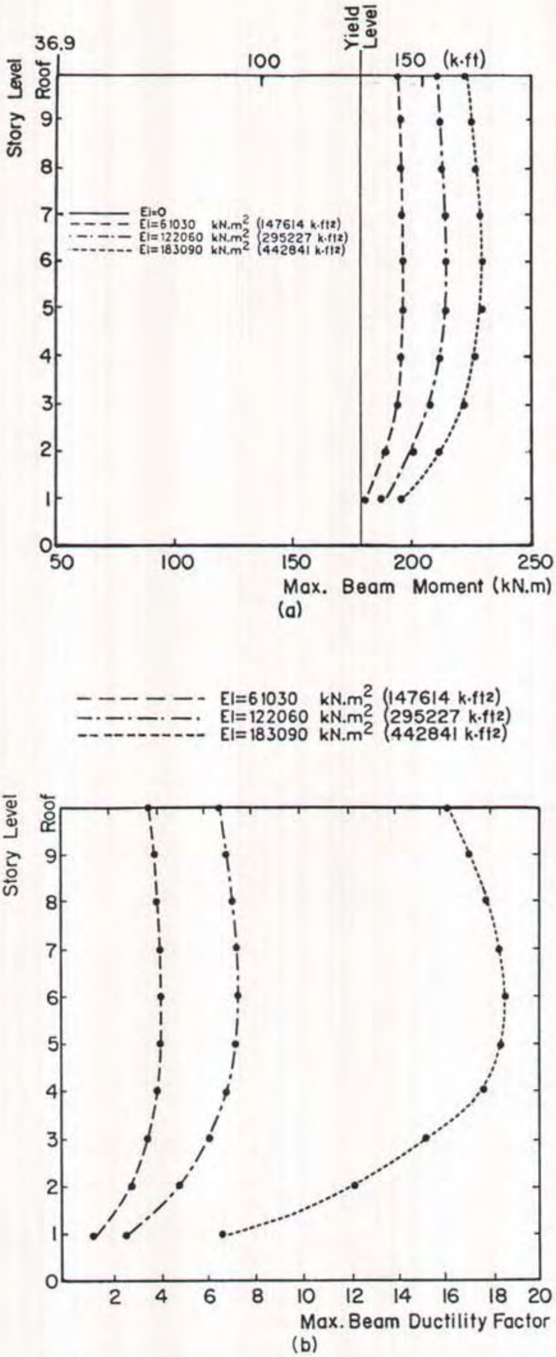


Fig. 15. Maximum beam moments and maximum beam ductility factor showing the effect of beam stiffness: slender beams, post-tensioning for vertical continuity.

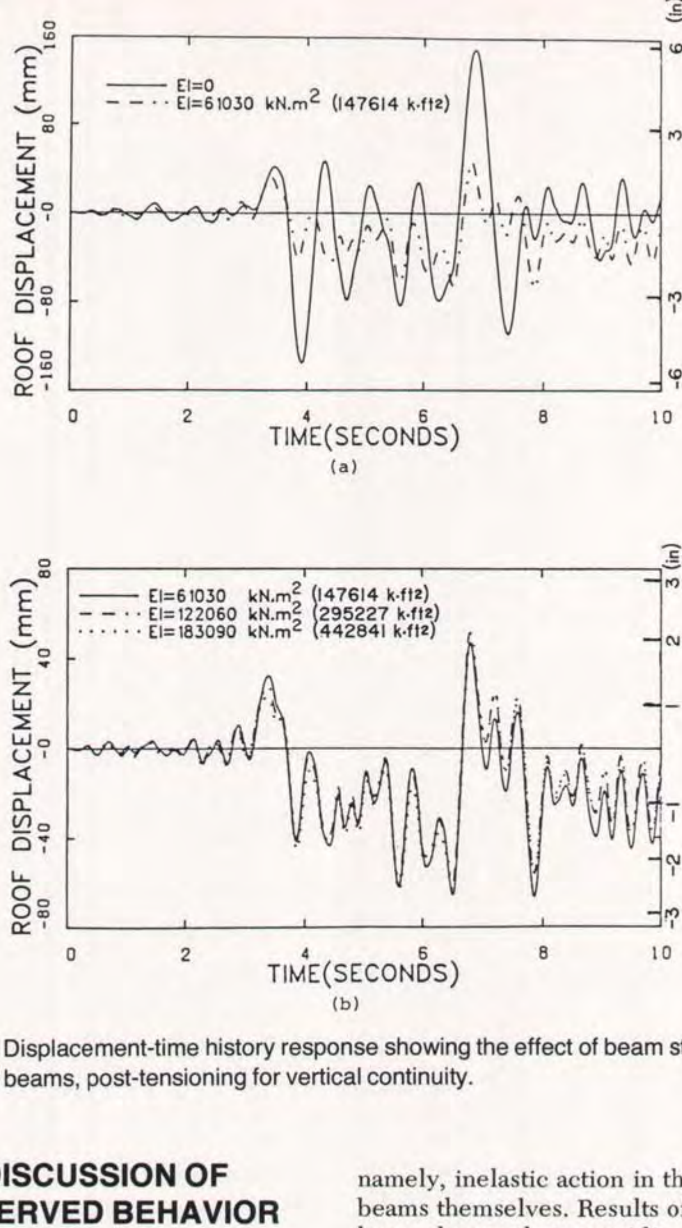


Fig. 16. Displacement-time history response showing the effect of beam stiffness: slender beams, post-tensioning for vertical continuity.

DISCUSSION OF OBSERVED BEHAVIOR

This study and previous studies have shown that energy dissipation in simple precast wall systems under earthquake loading is a combination of slip along horizontal connections and rocking at horizontal joints. The introduction of coupling beams allows a third mechanism for energy dissipation, namely, inelastic action in the coupling beams themselves. Results of this study have shown that providing coupling beams can significantly improve behavior by suppressing the rocking mechanism and thereby reducing the likelihood of a high concentration of stress and strain in the horizontal connections.

It has also been shown that behavior is significantly affected by the method of

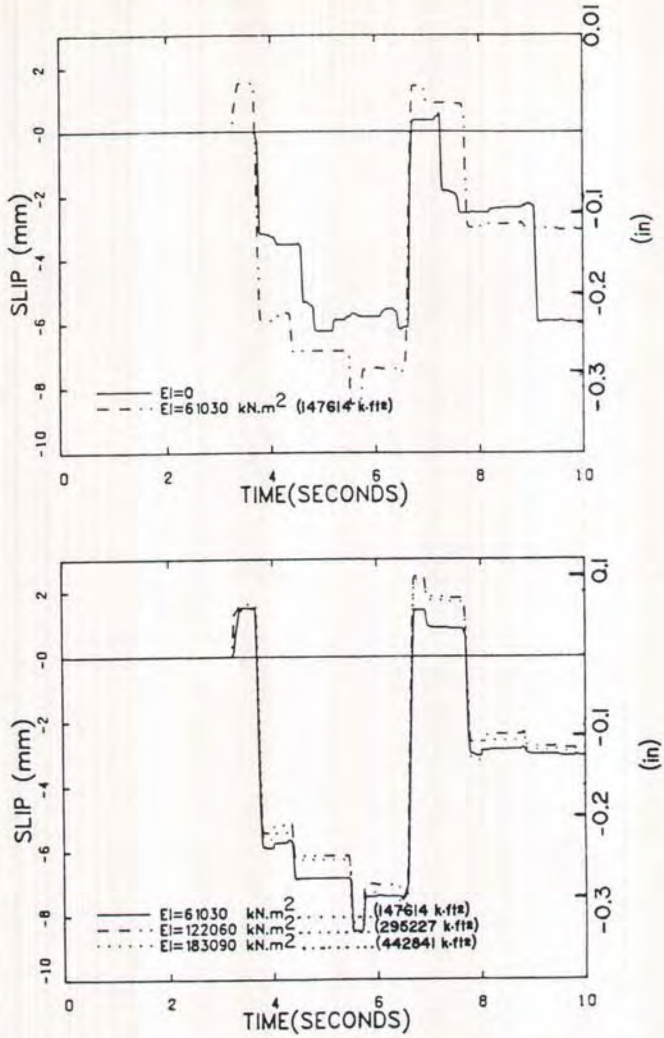
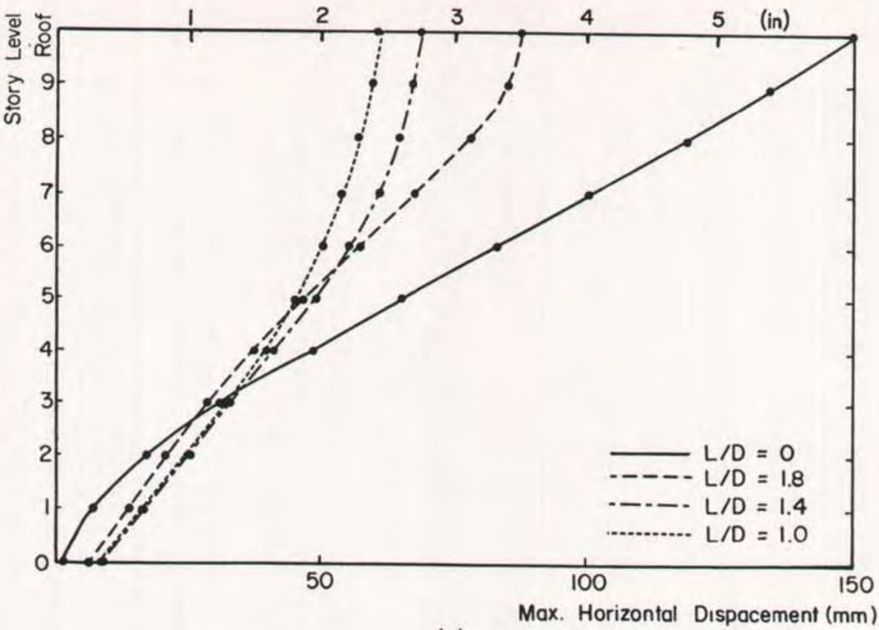


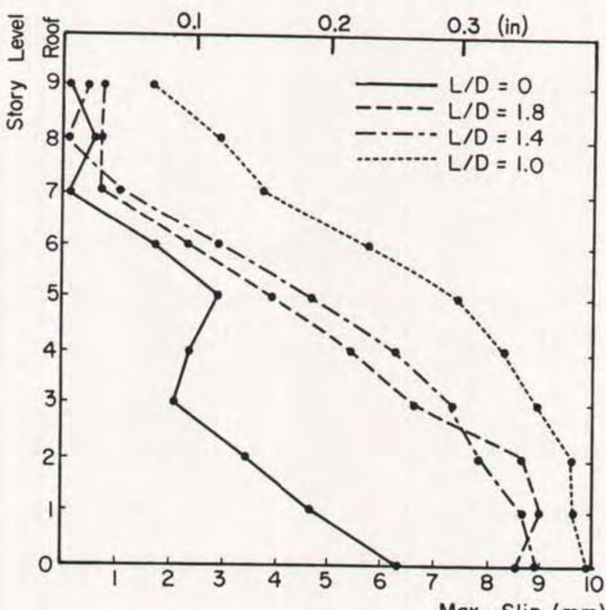
Fig. 17. Slip-time history response showing the effect of beam stiffness: slender beams, post-tensioning for vertical continuity.

providing vertical continuity. When mild reinforcement is used, the shear-friction mechanism limits the tendency for slip along horizontal connections, thus producing a greater tendency for rocking. For the earthquake intensity selected for this study, structures with this type of vertical continuity were unable to survive the full 10 seconds of earthquake motion.

Further studies are necessary to clearly define the limits of applicability of coupled walls with mild reinforcement across the horizontal joints. While the post-tensioned walls did produce better all-round performance, in many cases the accompanying slip values were quite large. Such behavior is undesirable because of the potential for overall instability and the fact that with large slippage occurring, the structure is unlikely to return to its original unde-

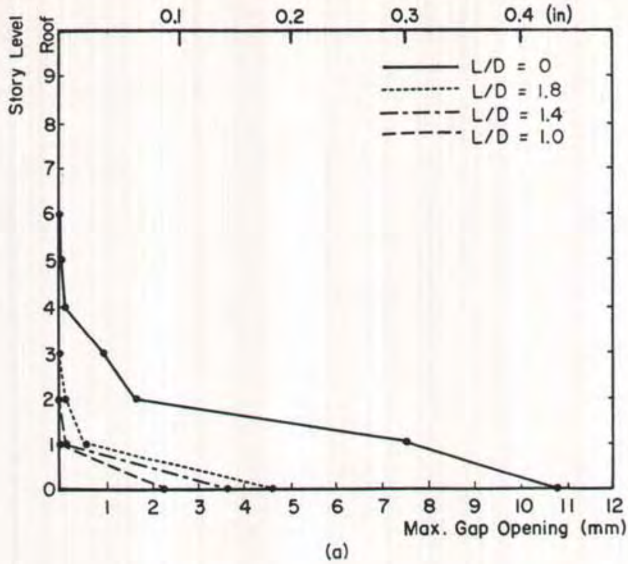


(a)



(b)

Fig. 18. Displacement and shear slip envelopes showing the effect of beam stiffness: deep beams, post-tensioning for vertical continuity.



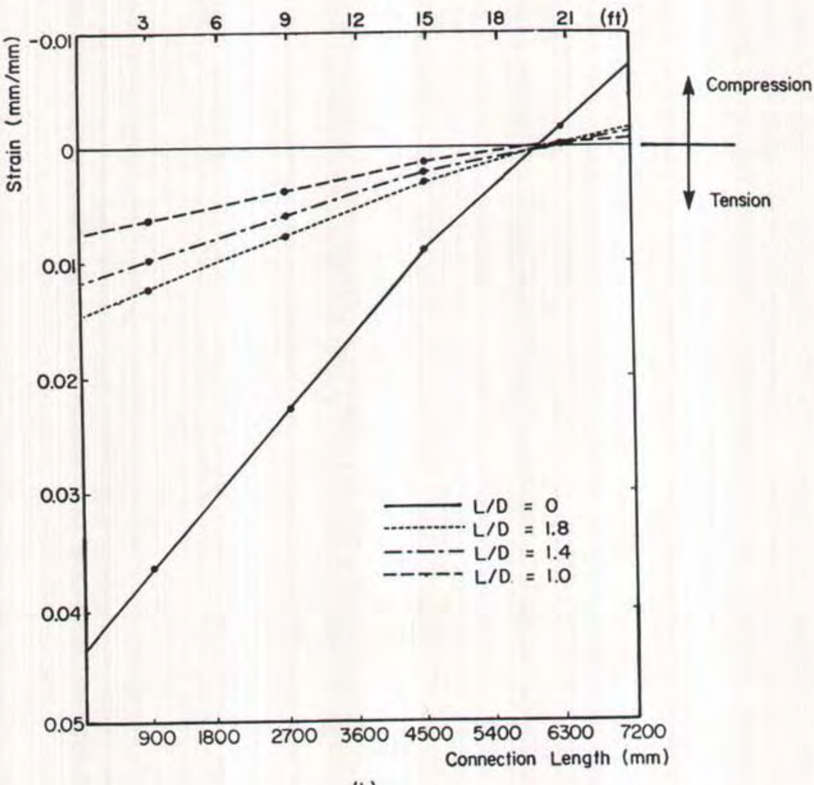


Fig. 19. Gap opening and strain distribution showing the effect of beam stiffness: deep beams, post-tensioning for vertical continuity.

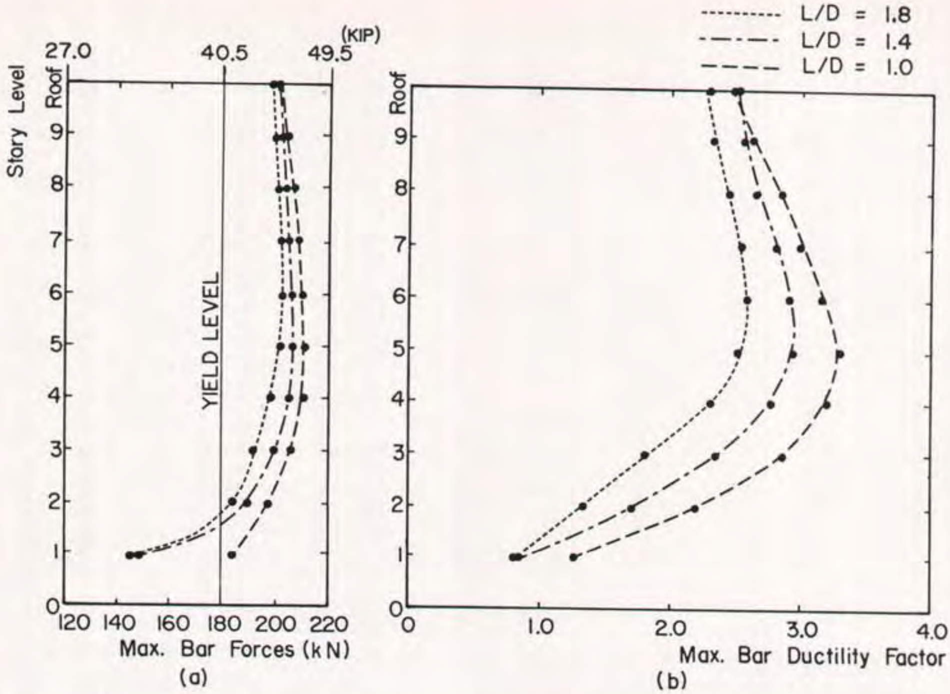


Fig. 20. Maximum bar forces and maximum bar ductility factor showing the effect of beam stiffness: deep beams, post-tensioning for vertical continuity.

formed postion at the end of the earthquake motion.

IMPLICATIONS FOR CONSTRUCTION AND DESIGN

The results of this study indicate that coupling beams can be used to improve seismic response of large panel systems. However, a number of practical problems need to be addressed.

Construction

Coupling beams can be cast monolithically with the panels in the precast plant to provide essentially the same arrangement as used in cast-in-place construction. In fact, this may be the only practical approach to incorporating the diagonal truss reinforcing details required for deep beams. The size and shape of the combined precast unit may require the use of special frames to support the unit to prevent damage during transportation and erection.

For slender beams, each half of the beam could be cast integrally with the adjacent wall panel as suggested by Mueller.[19] A shear transfer joint could then be incorporated at midspan of the coupling beam.

Steel or precast beams could also be used if ductile moment-resisting connections between beam and panel can be provided. Bhatt and Kirk[20] tested welded beam-column connections for precast concrete elements under cyclic loading. They showed that adequate strength and ductility can be obtained, although tolerance requirements are quite severe.

Additional laboratory tests would be useful to develop practical and economical connection details, and thus provide the required strength and ductility characteristics.

Design

Mueller[19] has identified four possible approaches for the design of large panel wall systems for seismic loading:

(a) Monolothic design
(b) Elastic limit design
(c) Weak horizontal joint design
(d) Weak vertical fiber design

An optimum design for earthquake loading is likely to involve a combination of energy dissipation in coupling elements (beams or connectors) with some controlled amount of inelastic action permitted in the horizontal joints. Further studies will be required to develop a sound basis for design along these lines. Laboratory tests are needed to develop practical coupling beam details to ensure that the required ductility is obtained. For example, the detailing of bends in diagonal bars to avoid horizontal joints requires careful attention.

It is also conceivable that design could be based directly on dynamic inelastic analysis of the type described in this paper if appropriate earthquake and structural characteristics, as well as acceptable levels of slip at horizontal joints, can be defined.

Recently, Clough[21] has proposed a practical design approach that considers explicitly both strength and inelastic deformation of members. Dynamic inelastic analyses of the type described in this paper, along with results of laboratory tests, could be used to verify and refine Clough's design procedure.

CONCLUSION

The structures considered in this study have been subjected to very high earthquake intensity to observe general trends in behavior. The results suggest that both slender coupling beams and deep coupling beams provide beneficial effects on overall behavior. While a post-tensioned wall system appears to provide the best overall performance in terms of surviving the earthquake, the tendency to produce large slip between adjacent panels is undesirable, and it is possible that at lower earthquake intensity the provision of mild steel reinforcement for vertical continuity might produce a more desirable response.

The present study has considered a limited range of structural configurations, analytical modeling parameters and earthquake characteristics. Further studies would be helpful to delineate the range of applicability of these types of structures for seismic zones and the optimum set of structural parameters for given design situations. The analytical results of this study are based on relatively simple and crude models of precast coupling beams. Laboratory tests of coupling beam arrangements are required before attempting to develop more elaborate models.

While the dynamic inelastic computer modeling techniques employed may be considered too refined for general design use, it is believed that they could be a useful tool for special cases.

ACKNOWLEDGMENT

Financial assistance for this study was provided by the Natural Sciences and Engineering Research Council of Canada through Operating Grant A5153.

* * *

REFERENCES

1. Fintel, M., "Performance of Precast Concrete Structures During Rumanian Earthquake of March 4, 1977," PCI JOURNAL, V. 22, No. 2, March-April 1977, pp. 10-15.
2. Englekirk, R. E., "Overview of Workshop on the Effective Use of Precast Concrete for Seismic Resistance," PCI JOURNAL, V. 31, No. 6, November-December 1986, pp. 48-58.
3. Oliva, M. G., and Shahrooz, B. M., "Shaking Table Tests of Wet Jointed Precast Wall Panels," Proceedings of the Eighth World Conference on Earthquake Engineering, San Francisco, California, V. VI, 1984, pp. 717-724.
4. Velkov, M., Ivkovish, M., and Perishich, Z., "Experimental and Analytical Investigation of Prefabricated Large Panel Systems to be Constructed in Seismic Regions," Proceedings of the Eighth World Conference on Earthquake Engineering, San Francisco, California, V. VI, 1984, pp. 773-780.
5. Harris, H. G., and Caccese, V., "Seismic Behavior of Precast Concrete Large Panel Buildings Using a Small Shaking Table," Proceedings of the Eighth World Conference on Earthquake Engineering, San Francisco, California, V. VI, 1984, pp. 757-764.
6. Becker, J. M., and Llorente, C., "The Seismic Response of Simple Precast Concrete Panel Walls," Proceedings of the U.S. National Conference on Earthquake Engineering, Stanford University, Stanford, California, 1979.
7. Schricker, V., and Powell, G. H., "Inelastic Seismic Analysis of Large Panel Buildings," Report No. UCB/EERC-80/38, College of Engineering, University of California, Berkeley, 1980.
8. Chu, Y., Liu, Y., Chen, R., Guan, Q., and Shou, G., "Experimental Study on the Seismic Behavior of Multistory Precast Large Panel Residential Buildings," Proceedings of the Eighth World Conference on Earthquake Engineering, San Francisco, 1984, V. VI, pp. 781-788.
9. Paulay, T., and Santhakumar, A. R., "Ductile Behavior of Coupled Shear Walls," *Journal of the Structural Division*, ASCE, V. 102, 1976, pp. 93-108.
10. Paulay, T., and Binney, J. R., "Diagonally Reinforced Coupling Beams of Shear Walls," *Shear in Reinforced Concrete*, Special Publication SP-42, American Concrete Institute, 1974, pp. 579-599.
11. Saatcioglu, M., "Inelastic Behavior and Design of Earthquake Resistant Coupled Walls," PhD Thesis, Northwestern University, Evanston, Illinois, 1981.
12. Takayanagi, T., Scanlon, A., and Corley, W. G., "Earthquake Resistant Structural Walls—Analysis of Coupled Wall Specimens," Report to National Science Foundation by Portland Cement Association, Skokie, Illinois, 1981.
13. Kianoush, M. R., and Scanlon, A., "Analytical Modeling of Large Panel Coupled Wall Systems for Earthquake Loading," *Canadian Journal of Civil Engineering*, August 1988.
14. Kanaan, A. E., and Powell, G. H., "A General Purpose Computer Program for Inelastic Dynamic Response of Plane Structures," Report No. EERC 73-22, University of California, Berkeley, 1975, 101 pp.
15. National Building Code of Canada, National Research Council of Canada, Ottawa, 1985.
16. Kianoush, M. R., and Scanlon, A., "Inelastic Seismic Response of Precast Concrete Large Panel Coupled Shear Wall Systems," S.E.R. No. 134, University of Alberta, Edmonton , March 1986, 300 pp.
17. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-77)," American Concrete Institute, Detroit, 1977.
18. Earthquake Engineering Research Institute, Report No. EERI 76-02, California Institute of Technology, Pasadena, 1976, 77 pp.
19. Mueller P., "Behavioral Characteristics of Precast Walls," Proceedings, Workshop on Design of Prefabricated Concrete Buildings for Earthquake Loads, ATC-8, Applied Technology Council, Berkeley, California, 1981, pp. 277-308.
20. Bhatt, P., and Kirk, D. W., "Tests on Improved Beam-Column Connection for Precast Concrete," *ACI Journal*, Pro-

ceedings V. 82, No. 6, November-December, 1985, pp. 834-843.

21. Clough, D. P., "Design of Connections for Precast Prestressed Concrete Buildings for the Effects of Earthquake," Technical Report No. 5, Prestressed Concrete Institute, Chicago, Illinois, March 1985.

* * *

NOTE: Discussion of this paper is invited. Please submit your comments to PCI Headquarters by June 1, 1989.

APPENDIX A — ANALYTICAL MODELING

Several models were incorporated into the computer program DRAIN-2D[14] which allows for inelastic action using piece-wise linear force-deformation relationships. A linear elastic plane stress rectangular element was incorporated to model wall panel behavior. Inelastic action was confined to horizontal connections and coupling beams.

Horizontal Connections

Models proposed by Becker and Llorente[6] were incorporated with slight modification to suit the piece-wise linear formulation in DRAIN-2D. Various analytic models depicting force-deformation characteristics of connection elements are shown in Figs. Al, Parts (a) through (d).

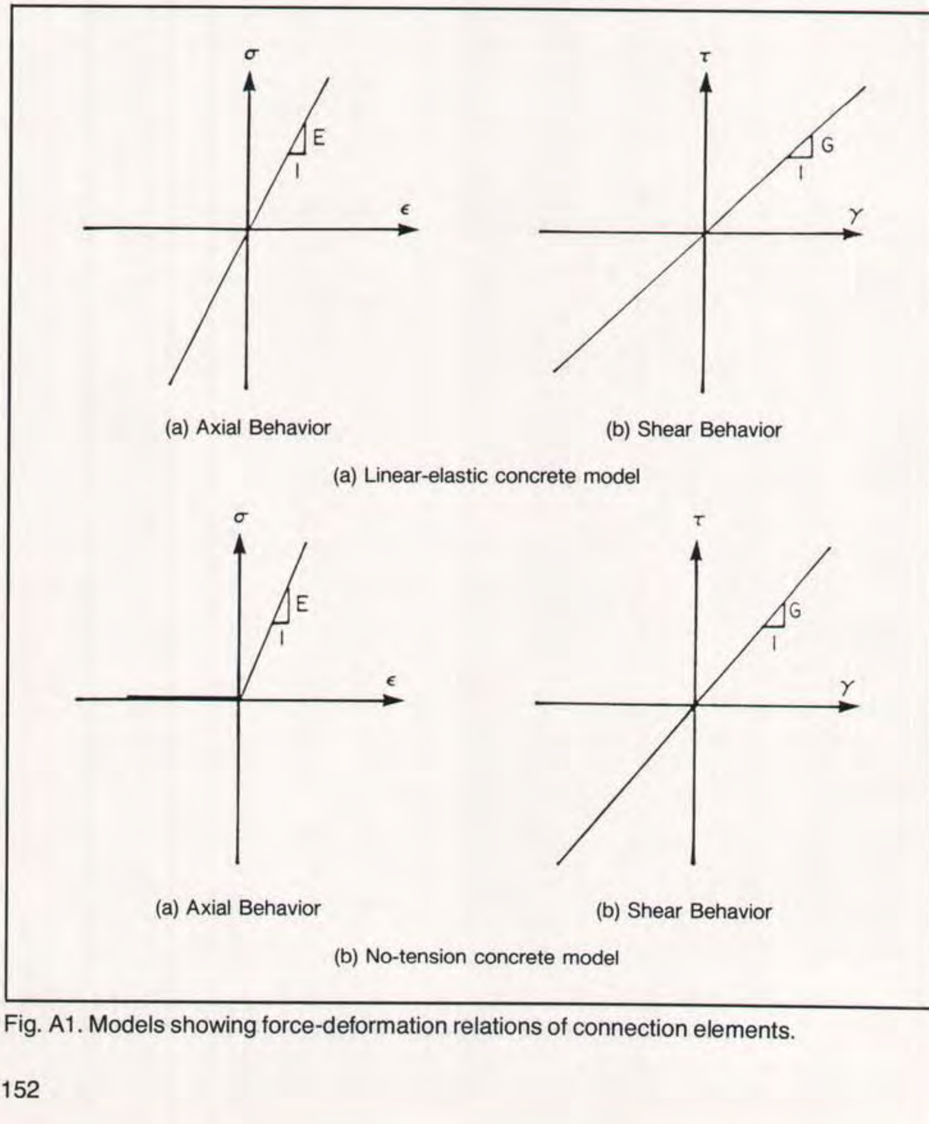


Fig. A1. Models showing force-deformation relations of connection elements.

The multilinear concrete model was used in the parametric study. Post-tensioning bars were assumed to remain linear elastic and to be ungrouted, extending from roof to foundation.

Stress-strain models for shear slip and shear-friction are shown in Figs. 11 and 12 of the paper.

Coupling Beams

Behavior of coupling beams was modeled using existing DRAIN-2D elements. The DRAIN-2D reinforced concrete beam model was used for slender beams while the inelastic truss element was used to model diagonal bars of deep beams in tension and compression.

* * *

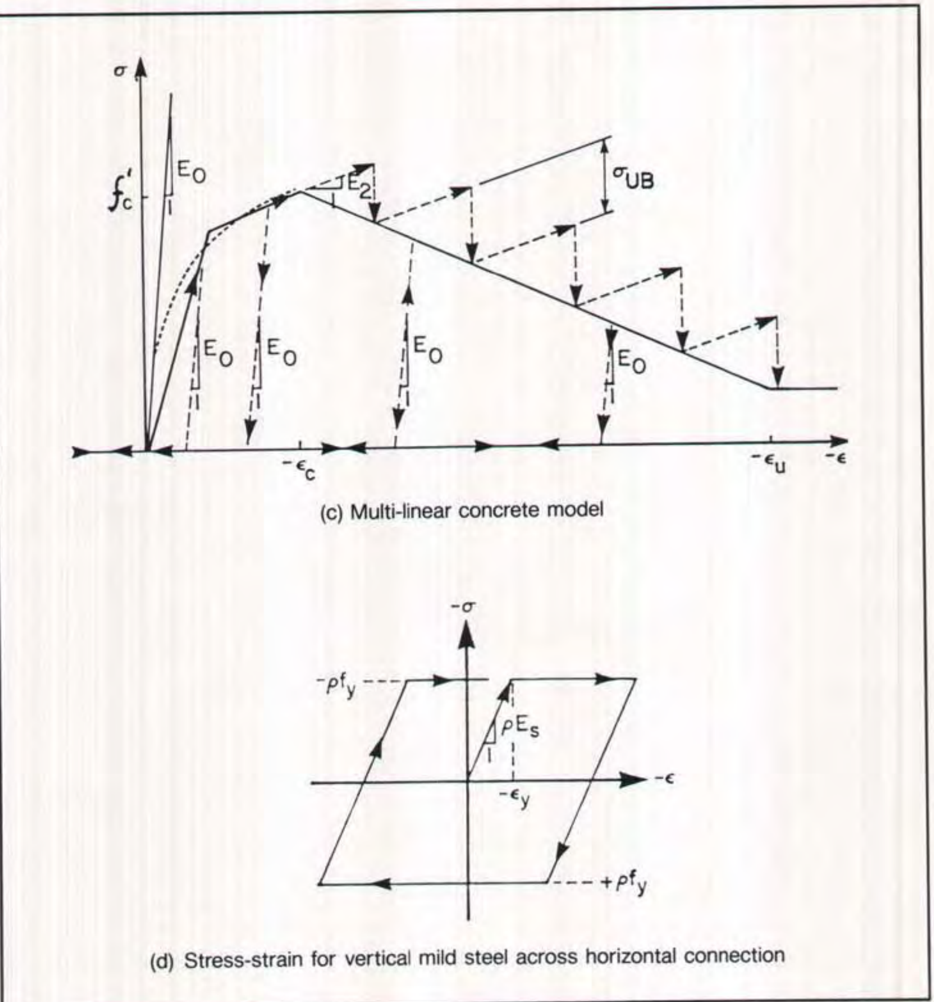


Fig. A1 (cont.). Models showing force-deformation relations of connection elements.