THE SEISMIC RESPONSE OF SIMPLE PRECAST CONCRETE WALLS

James M. Becker[I] Carlos Llorente[II] Peter Mueller[III]

SUMMARY

This paper presents a series of analytic studies that assess the seismic response of simple precast concrete walls. A finite element idealization along with an explicit time-step integration scheme was used for the inelastic dynamic analysis. These studies examine the response characteristics for various building configurations and connection characteristics. Of particular importance has been the role of vertical continuity across the horizontal connection in terms of both compressive and tensile force transfer. The role of friction in shear transfer is also explored, and the possible existence of a global slip phenomenon is examined.

SIMPLE PRECAST CONCRETE WALLS

A simple precast concrete wall is a stack of large precast concrete panels and thus has only horizontal connections. This type of wall is a basic lateral load resisting element in the cross wall configurations commonly found in the United States and Canada. It should also be noted that the simple wall response represents a bound for the seismic response of composite walls (see Fig. 1), that is, walls joined into composite shapes through vertical connections.

The cross wall systems mentioned above are characterized by the use of one way precast prestressed floor planks spanning between the load-bearing simple walls. The resulting horizontal connection is of the platform type and is illustrated in Figure 2. The manner in which this connection transfers forces between the precast wall panels is critical to the overall behavior of the simple wall. In general the connection acts as a plane of reduced strength and stiffness as can be seen from the limited experimental data [7,8] available on its compressive strength.

Many techniques have been developed for facilitating the necessary connection of vertical reinforcement across the horizontal connection: however, few, if any, allow for the economic development of the amount of vertical steel normally expected in earthquake resistant cast-in-place shear walls. Because of this minimum amount of vertical reinforcement and the effects of shrinkage and creep, it is expected that cracking will be concentrated in the horizontal connections. Thus as lateral forces are applied and reversed, cracks will open and close along the horizontal connections. This phenomenon is usually referred to as rocking (see Fig.3).

The transfer of shear along the horizontal connection may be accomplished by the following mechanisms: (a) coulomb friction, μN, (b) shear friction, associated with transverse reinforcement across the crack, $\mu A_s f_y$, and (c) by mechanical means such as keying and metal details. The first two mechanisms may be accompanied by significant shear displacement (slip) and

[I] Associate Prof. of Civil Engineering, Mass. Institute of Technology.
[II] Research Assistant, Dept. of Civil Engrg., Mass. Institute of Technology.
[III] Research Associate, Dept. of Civil Engrg., Mass. Institute of Technology.

may exhibit ductile behavior [6,9,12]. The latter mechanism requires smaller shear displacements and may exhibit ductile or brittle behavior depending on the detail used.

It is typical to assume that the ultimate shear strength of a connection, considering only friction transfer mechanisms, is the product of the friction coefficient, μ, and the sum of the normal force, N, and the steel yield force, $A_s f_y$. However, it should be noted that the latter contribution ($\mu A_s F_y$) to the ultimate strength in shear can only be developed when there is relative slip in the cracked plane. Thus the shear resistance is initially given by the coulomb friction component (μN), which requires no relative slip to be active. In general, the lower bound for the friction coefficient is considered to be around 0.7 [1,12]. However, under the unique conditions of smooth panel edges and grouted or dry packed interfaces, along with cyclic loading, a lower value may be expected [9]. Values as low as 0.4 [4] and even 0.2 [6] have been suggested.

Regardless of the particular shear transfer mechanism, global shear slip along the entire length of the connection may be possible (Fig.3). It should be noted that such global slip represents an unconfined yield mechanism when only coulomb friction is involved. The actual displacement is only controlled by reversals of acceleration or by the stiffness of secondary elements if present.

SEISMIC RESPONSE

A series of parametric studies have been carried out to examine the influence that the previously discussed connection characteristics may have on seismic response. To carry out these studies, a finite element idealization was used along with an explicit time-step integration scheme. The large panels were assumed to remain linear elastic and were thus modeled as statically condensed super elements (see Fig. 4). The connection regions were modeled with contact elements with two integration points. This modeling technique and the parametric studies to be discussed have been reported in greater depth in Ref. 2 and 3.

To carry out the parametric studies, a cross wall building was identified and typical panel dimensions and floor masses were determined [5]. In order to isolate a simple wall from the building it was assumed that both the floor diaphragm and the foundation were rigid. In this manner five and ten story simple walls (see Fig. 5) were identified. The wall panels had an effective modulus of 25.2×10^6 kPa ($\upsilon = 0.15$) and the connection modulus in compression was one half that of the wall panels. Tension was assumed to be transferred through the reinforcement only. The reinforcement was modeled as an elasto-plastic spring whose initial stiffness was determined by assuming the bar to be unbonded in the connection region. Only coulomb friction was considered for shear transfer along horizontal joints and elasto-plastic hysteretic behavior was assumed with a secondary stiffness of one hundredth that of the connection's primary shear stiffness.

Six different cases of vertical continuity across the horizontal connection were considered. A first case assuming linear elastic behavior serves as a reference for response comparisons. For this baseline case the ten-story structure had a fundamental period of 0.62 seconds, while the five-story structure had a fundamental period of 0.17 seconds. The remaining five cases refer to nonlinear and inelastic connection characteristics. Two cases have vertical continuity provided by ungrouted post-tensioned bars, but have different friction coefficients (μ = 0.2 and 0.4). In both the 5-and 10-story walls a uniform prestress of 1400 kPa was applied. Three cases have vertical continuity provided by Grade 40 reinforcement, either evenly distributed along each connection level (0.25% and 1.00%), or concentrated at the ends of the panel (0.5%). All three reinforced cases have the same friction coefficient (μ = 0.4).

The majority of the parametric runs were made using an artificial earthquake generated to match the Newmark-Blume-Kapur response spectra for 2% damping. To provide a comparison for these results, several analyses were made using the El Centro (N.S. Component, 1940) and Taft (Kern County N21E, 1952) records. The response spectra of all three earthquakes for 5% critical damping (which was used in the analyses) are given in Figure 6.

Figure 7 presents three different cases that are intended to illustrate the basic characteristics of both the rocking and the shear slip phenomena. All three cases refer to the ten-story wall being subjected to the artificial earthquake with a peak acceleration of 0.30g. Case 1 is the linear elastic reference case. In this case the maximum base shear was 3954 kN. As was to be expected, the axial strain and shear stress distribution corresponds to normal beam theory. The axial strains and shear stresses are given for the time of maximum overturning moment.

In Case 2 tension across the connection is solely transferred through ungrouted posttensioning bars. The shear friction coefficient is 0.4. The maximum base shear has dropped to a value of 2476 kN. This decrease in maximum base shear is attributable to an elongation in the apparent fundamental period associated with the rocking of the wall and an ensuing shift in the response spectra. This elongation of the fundamental period is easily observed by comparing the time histories of Case 1 and Case 2. In the tension regions the axial strain is obtained by dividing the crack opening by the height of the connection. As can be easily seen in Fig. 7, during rocking, plane sections no longer remain plane in the region of the connection. According to the coulomb friction mechanism, shear is now transferred only in the unopened portion of the connection.

If vertical continuity is provided by ungrouted post-tensioned bars, and within the range of the parameters explored, rocking is basically a nonlinear, elastic phenomenon as illustrated by Case 2. In all cases, for which Figure 8 gives the extent of crack opening, the concrete axial strains, remained in a region within which the assumption of linear elastic behavior was reasonable. The forces in the ungrouted post-tensioning bars did not approach the yield strength because of the extreme flexibility of ungrouted bars. In the mildly reinforced 10-story walls, however, the reinforcing yielded.

The magnitude of the friction coefficient, together with the compressive resultant due to gravity loads, post-tensioning (if present), and the stressing of mild reinforcement (if present), determine whether global shear slip will occur. The effect of global slip is to limit seismic force levels and to dissipate energy as has been previously noted [4,11]. The global slip phenomenon is illustrated by Case 3, which is the same as Case 2 except that the coefficient of friction, μ, has been lowered to 0.2. Note how the shear stress distribution now directly follows the axial stress distribution.

Figure 8 presents the envelopes of maximum global slip for a series of the parametric runs. The post-tensioned cases with the lower friction coefficient (μ = 0.2) experience global slip along all connection levels. It is interesting to note that in both Fig. 8a and 8c the peak global slips for the 10 story μ = 0.2 cases are located from the third through the fifth levels. This can be attributed to a combination of higher mode contributions and a decreasing shear strength with height. No slippage was observed in any of the post-tensioned, μ = 0.4, cases. However in all of the regularly reinforced cases global slippage was observed in the upper stories. The magnitude of this global slip indicated that mild reinforcement will be engaged in developing a shear friction mechanism and that this should be accounted for in future modeling.

In Figure 9 the base shear and cumulative base shear slip time histories are presented for the 10-story post-tensioned (μ = 0.2) case subjected to the El Centro earthquake (0.25g). Figure 10 presents the flow of principal stresses in the bottom two panels of this case at an instant of global slip. As can be seen in this figure, the confluence of shear and compressive forces leads to a thrusting action that can create severe biaxial stress conditions in the connection as well as in the panel corners. Of particular concern is the upper corner of the panel, in which significant tensile stresses are found, indicating the need for care in the design of panel reinforcement. The need to avoid this force concentration has caused some designers to recommend the use of shear keys in the horizontal connection (10).

CONCLUSIONS

The seismic response of simple precast walls is found to be mainly governed by a rocking phenomenon. The opening of the horizontal connection, associated with rocking, leads to a progressive softening of the structure with increasing excitation level. Thus, rocking leads to an elongation of the apparent fundamental period of the wall, with a consequent increase or decrease in the seismic response, depending upon the nature of the earthquake.

It was found that within the parameters explored global slippage between two panels would occur only in limited situations. These situations included extremely low coefficients of friction, the upper levels of non-posttensioned walls and shorter walls. It was concluded that global slippage could not normally be counted upon for seismic force isolation.

Rocking coupled with local and global shear slip leads to significant force concentrations in both the corners of panels and the ends of connections. It is felt that such force concentrations, unless specifically designed for, will lead to a progressive deterioration of the panel corners and connection ends in a significant earthquake. The concern for these force concentrations, along with a hesitancy to use an unconfined yield mechanism to limit forces, suggests caution in relying on beneficial effects of global shear slip mechanisms in the design of simple walls.

The work presented in this paper is based on the limited experimental data available about the behavior of the panelized buildings. This data calls for simple and therefore also questionable assumptions with respect to material behavior. Accordingly, the computer studies presented are limited in their scope. However, it is felt that the basic behavioral aspects presented are correct and point to some of the major problems in the seismic design of panelized buildings.

ACKNOWLEDGEMENTS

This research on the Seismic Resistance of Precast Concrete Panel Buildings is being conducted at the Massachusetts Institute of Technology (Cambridge, Massachusetts, U.S.A., 02139) and has been sponsored by the National Science Foundation under Grant ENV 75-03778 and Grant PFR-7818742. In addition, the authors would like to acknowledge the assistance of Joseph Burns, and Maria Kittredge in the preparation of this paper.

REFERENCES

(1) Becker, J.M., and Llorente, C., "Seismic Design of Precast Concrete Panel Buildings," Proceedings of a Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, Vol. 3, Univ. of California, Berkeley, July, 1977, pp.1753-1798.

(2) Becker, J.M. and Llorente, C., "The Seismic Response of Simple Precast Concrete Panel Walls," Proceedings of the 2nd U.S. National Conference on Earthquake Engineering, Stanford, Cal., August, 1979, pp. 423-432.

(3) Becker, J.M., Llorente, C., and Mueller, P., "Seismic Response of Precast Concrete Walls," presented at the 11th Joint Meeting of the UJNR Panel on Wind and Seismic Effects, Tsukuba, Japan, Sept., 1979 and to be published in The Journal of the International Association of Earthquake Engineering.

(4) Brankov, G., and Sachanski, S., "Response of Large Panel Buildings for Earthquake Excitation in Nonelastic Range," Proceedings of the Sixth World Conference on Earthquake Engineering, New Delhi, India, 1977.

(5) Frank, R., "Dynamic Modeling of Large Precast Panel Buildings Using Finite Elements with Substructuring," Report No. R76-36, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, Massachusetts, August, 1976.

(6) Hanson, N.W., "Seismic Test of Horizontal Joints," Design and Construction of Large Panel Concrete Structures, Supplemental Report C., prepared for U.S. Dept. of Housing & Urban Development, PCA. January, 1979.

(7) Iyengar, S., and Harris, H.G., "Behavior of Horizontal Joints in Large Panel Precast Concrete Buildings," Report No. M78-1. Structural Models Laboratory, Dept. of Civil Eng., Drexel Univ., Philadelphia, Penn., 1978.

(8) Joahl, L.S., and Hanson, N.W., "Horizontal Joint Tests," Design and Construction of Large Panel Concrete Structures, Supplemental Report B., prepared for U.S. Dept. of Housing & Urban Development, PCA, Nov., 1978.

(9) Mattock. A.H., "Shear Transfer Under Cyclically Reversing Loading, Across an Interface between Concrete Cast at Different Times," Final Report Part 2, Dept. of Civil Eng., Univ. of Washington, Seattle, Wash., June, 1977.

(10) Polyakov, S., Design of Earthquake-Resistant Structures, 1st ed., MIR Publishers, Moscow, 1974.

(11) Powell, G., and Schricker, V., "Ductility Demands of Joints in Large Panel Structures," ASCE Fall Conv., San Francisco, Oct., 1977, Preprint 3022.

(12) Verbic, B., "Behavior of Joint Elements of a Building Constructed of Large Panels by the System 'Vranica' Under Quasistatic Cyclic Loading," Institute za Materijale i Konstruckcije, Sarajevo, April, 1977.

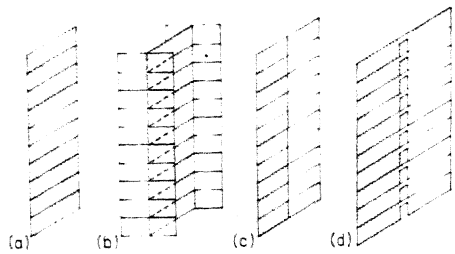

FIGURE 1 PRECAST WALL CONFIGURATIONS (Shading denotes a simple wall)

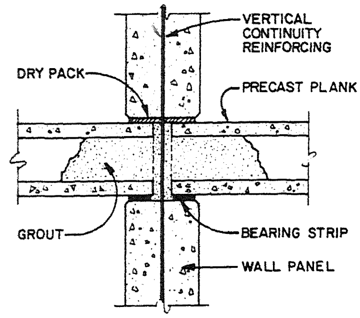

FIGURE 2 TYPICAL PLATFORM CONNECTION

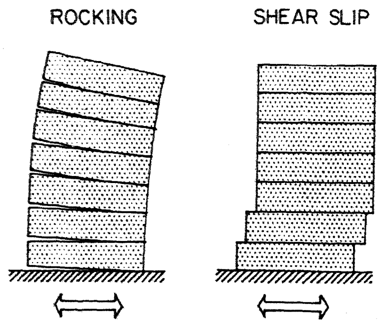

FIGURE 3 ROCKING AND SHEAR SLIP MECHANISMS

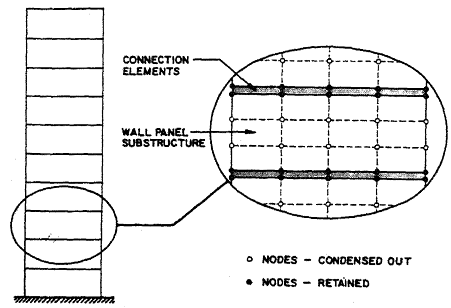

FIGURE 4 FINITE ELEMENT MODEL

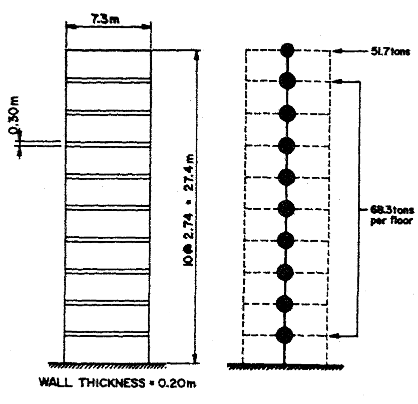

FIGURE 5 10-STORY SIMPLE WALL

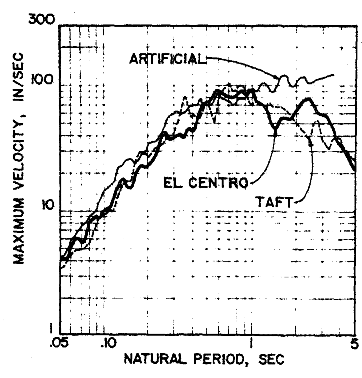

FIGURE 6 RESPONSE SPECTRA, 5% DAMPING, 1.0g PEAK ACCELERATION

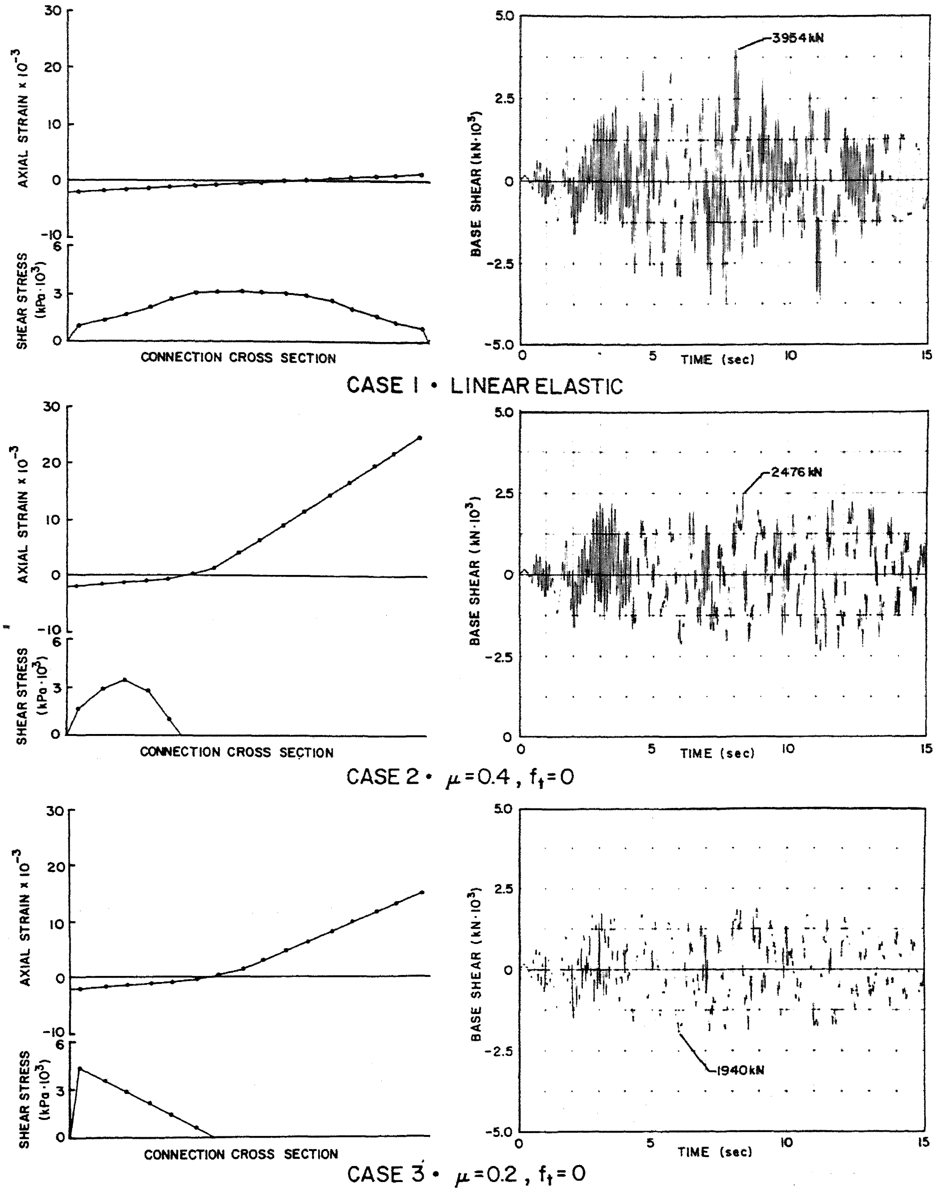

FIGURE 7 COMPARISON OF AXIAL STRAIN AND SHEAR STRESS AT TIME OF MAXIMUM CONNECTION OPENING, AND BASE SHEAR TIME HISTORIES, 10 STORY, P.T., ARTIFICIAL 0.30g.

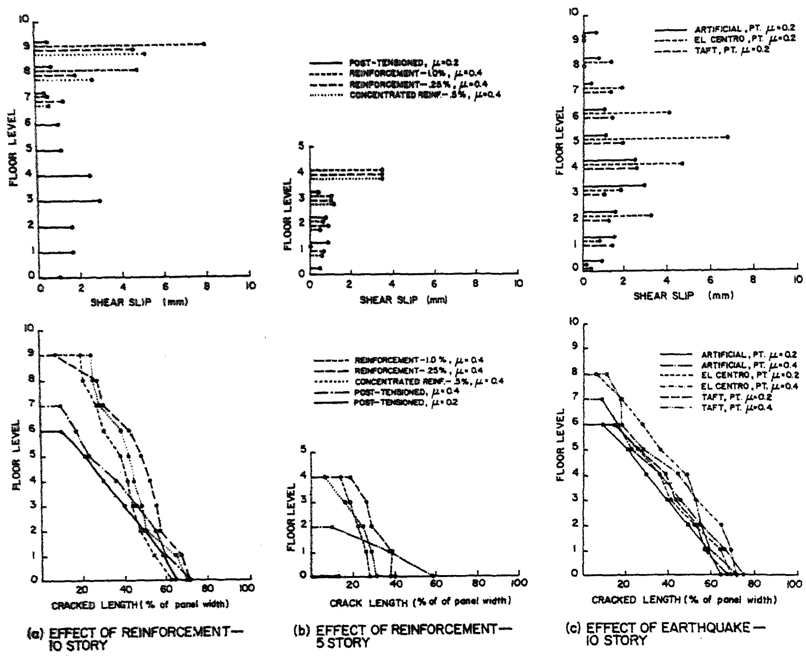

FIGURE 8 EFFECT OF REINFORCEMENT AND EARTHQUAKE ON SHEAR SLIP AND CRACK LENGTH (0.25g Peak acceleration)

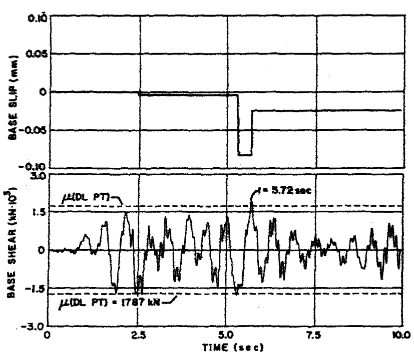

FIGURE 9 BASE SHEAR AND SLIP TIME HISTORIES AT BASE OF 10 STORY-WALL, P.T. $\mu = 0.2$, EL CENTRO 0.25g

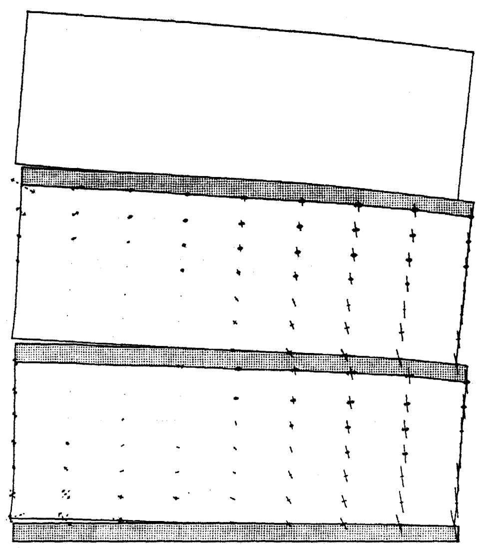

FIGURE 10 STRESS DISTRIBUTION AT BOTTOM OF 10 STORY WALL, AT t = 5.72 sec, P.T. $\mu = 0.2$, EL CENTRO 0.25g. (Arrows indicate tension).