Structures have traditionally been designed for earthquake resistance by providing a sufficient strength level to resist a prescribed static lateral load. Though previous codes only defined the required lateral loading, there was an implicit assumption that a certain amount of post-yield deformability would be available within the structure while a yield level resisting capacity was maintained. Recent codes have specifically addressed the need for ductility to be provided during the design in conjunction with the specified design loading. It is unclear, however, what level of ductility might actually be available in precast concrete construction. These recent changes have left producers of precast components pondering the problem of how to provide a usable product and caused design engineers to become reluctant to design precast structures in seismic regions.

The competitive advantages of precast systems in building construction have created a demand for their use in seismic as well as nonseismic locations. Moreover, some areas of previous nonseismic use have recently been rezoned as seismic regions and seismic resistance design requirements will now often control over wind design criteria. The problem in design is that precast systems typically have weak connection regions. They cannot currently be easily joined in a manner which would resemble monolithic concrete and still be assembled efficiently. Their behavior...
under strong seismic excitation will generally be substantially different from monolithic systems, but their actual ductility level and failure sequence has not been well documented yet. The research program described here investigated the performance of one type of precast system, the panelized building system, under replicated seismic motion to determine the demand for ductility which would be produced and the failure condition which might develop in a wall of a prototype 13-story building.

Use of Precast Construction

Precast concrete has been used in markets around the world to satisfy the tremendous demand for housing by providing a rapidly built system using factory fabricated quality controlled components. The competitive edge gained by precast manufacturers in the United States and other countries has come from the development of refined modular building systems with standardized components and simple connections.

The panelized system is an example of a method which uses standardized wall and floor/roof precast panels connected together to form a complete box type structure without a separate framework. While individual panels in the system may be connected together by a variety of means, all the connection schemes tend to form points of weakness in the structure.

Seismic Resistance and Design of Precast Structures

Many of the precast systems in use throughout the world are not well suited to resist the force and deformation demands caused by earthquake loading. The systems were often originally designed for nonseismic regions, but their advantages led to later use in low rise buildings in seismically active areas. Now, high rise buildings are being built with precast component systems in the same active regions. The use of precast systems for lateral resistance in strong seismic areas has been precluded in most areas of the United States by code provisions which would require connections similar to monolithic cast concrete. The simplicity of the connections, which makes precast concrete economically viable, causes a lack of continuity in stiffness and concentrated deforma-

Synopsis

The demand for economically competitive precast building systems has been increasing around the world at the same time that concerns have grown regarding their seismic resistance capacities. Recent codes have rezoned portions of the United States to reflect higher seismic hazard and to require seismic resistant design where wind loading prevailed in the past. Designers have been left in the dilemma of having to provide seismic resistant design without having code provisions specifically addressing the unique characteristics of precast construction.

The seismic resistant capacities of one form of precast construction, the large panel wall system, are described in this report. An investigation consisting of shaking table tests with earthquake motion and subsequent analytical investigations of the seismic response are included.

A set of conclusions and suggestions for improved performance of precast large panel construction are given, based on the tests, analytical simulations, and results from other researchers, along with a detailed discussion of problems in precast concrete design and behavior of the precast concrete system.
tion demand in some locations may develop during an earthquake.

Precast panelized wall systems have a lack of continuity in the horizontal connections between vertical wall elements. The wall elements are relied upon to provide both vertical load carrying capacity and lateral load resistance as shear walls. Yet, the shear walls are characterized by cantilever beam type behavior with a lack of redundancy. Under lateral load the panel wall system’s ability to carry vertical loads may be jeopardized by the wall’s natural tendency to yield and deform inelastically within a few weak horizontal joints.

Monolithic construction of joints may be appropriate in strong seismic zones; however, in zones of lower seismicity, structures with weak joints may still perform satisfactorily. The present difficulty for design is in determining what relation between strength and ductility must be provided in such joints. Unfortunately, there is not a single answer to this design dilemma. For any given yield strength which a designer might provide within a joint, the earthquake energy level will dictate how much deformation capacity at the yield level (i.e., ductility) the joint must have available in order to perform without failure. If the earthquake transfers a specific amount of energy to the joint, that energy must be accommodated within the joint by either remaining elastic, with a large force and small deformation, or by yielding, with a lower internal force but larger deformation. This relation may be understood by viewing Fig. 1.

The relation described above is the approach used in PCI Technical Report No. 5, where a simplified approach to seismic resistant design of precast structures is outlined. The PCI method is not necessarily readily usable at this time, however, because it may be difficult to envisage a single-degree-of-freedom inelastic response mechanism. Thus, it could be hard to determine how much energy will be transferred into a structure, and more importantly, the actual yield capacity and available ductility in most common precast connections is not known yet. The yield level and available ductility must both be known to use the PCI approach or to assign a proper force reduction factor ($R$) in design by current code equivalent static load methods.
Objectives of Research Program

Shaking table tests would provide the best means to study the seismic behavior of precast systems, particularly the relation between provided strength and the demand for deformation induced by the earthquake. Static tests of individual components or joints can provide information on stiffness and strength, but not the amount of deformation which will be needed. The actual forces transferred into the structure and deformation developed during an earthquake, however, depend on the strength and ductility of the structure. The actual forces and deformation can only be determined by dynamic shaking tests or analyses. The first objective of this investigation was to quantitatively measure the response of a large scale panel wall system during an earthquake. There have been no previous shaking tests of large scale precast wall assemblages. The shaking table tests conducted in this program were intended to furnish a complete quantitative description of inelastic mechanisms and the effects of such mechanisms on the system's dynamic response.

The second objective of the research program was to test analytic methods which have been proposed for use in predicting the inelastic seismic response of large panel precast wall assemblages. Acceptable analysis methods could then be used to predict the response of a complete prototype wall system which would be too expensive and complex to test. Schrieker, Becker, and Khanoush developed computerized techniques based on static tests of wall connections and on assumed forms of system deformation for estimating inelastic response in precast wall systems. Shaking table experimental results combined with correlation studies would check the abilities of the existing programs and aid in improving analytic techniques. Analytic methods could then be used to develop improved designs by evaluating performance of structures whose joints have various yield strength-deformation limits.

Scope of Research Program

Three different 3-story assemblages of wall panels were tested under earthquake motion on the shaking table. Since the cost of shaking table testing limited the number of specimens which could be investigated, a few basic configurations had to be selected which would represent the most common types found in panel wall structures. Each specimen was a single wall section built at one-third scale to represent a portion of a 10 to 20-story precast shear wall. The measured test deformations in the walls were compared with computer aided predictions using various analytic models. These correlation studies indicated which specific inelastic mechanisms had to be accurately represented in the analytic model. The analytic model was then used to predict the behavior of a 13-story prototype wall system.

This report describes the basic research and development work which has been completed on large panel precast wall systems. The shaking table test program is outlined with an explanation of the model design, the testing system, the test procedures, damage observations and measured test results. The analytic correlation work is then summarized with a discussion of modeling techniques. Finally, knowledge gained from the tests and analytic work is used to predict likely seismic limit states for a 13-story prototype wall system.

LARGE PANEL PRECAST WALL SYSTEMS

Large panel building systems are composed of vertical wall panels which support horizontal roof and floor panels to form a box like structure as diagrammed in Fig. 2. The vertical panels...
are stacked and joined to create axial loadbearing shear walls while the horizontal panels act a diaphragms and gravity load collecting roof and floor systems. Martin, Patman and Zecck\textsuperscript{5,6,7} have assembled reviews of various basic large panel precast systems and the typical joint configurations.

The primary difference between the systems currently used is in the manner in which the vertical and horizontal panels are joined. Panel wall systems in the United States typically use a “platform” construction in which hollow-core slabs form a horizontal layer simply supported on the wall panels below. The upper wall panels then rest on the platform formed by the hollow-core floor slab as shown in Fig. 3. Panel systems in Europe and elsewhere, however, have connections which frequently use cast-in-place concrete to form the joint between the wall and floor panels. Nearly all systems suffer a similar weakness when used for resisting seismic loads, namely, economically efficient systems have only a limited amount of vertical reinforcement continuous across the joints and seismic loading creates shear and flexural demands which can easily exceed the capacity of that steel.

Earthquake Performance and Existing Data

The existing use of large panel precast systems was obviously accomplished with a considerable amount of testing and analysis. Fortunately, very few large panel buildings have ever been subjected to strong seismic motions. The lack of records of performance, however, has created uncertainty regarding their ability to withstand the large forces or energy levels which could result. The best account of large panel building performance may be found in the aftermath of the 1988 Armenian earthquake near Yeravan in the Soviet Union. The large panel building system used in Armenia was very similar to the system tested in this research project.

Information from disaster inspections has indicated that three of the hardest hit cities (Spitak, Leninakan and Kirovakan) experienced strong shaking over period ranges which would likely encompass the natural period of large panel buildings. Not a single large panel building, however, was categorized as having collapsed or been damaged to a degree requiring demolition though a number of 4 to 9-story large panel structures existed.\textsuperscript{8} In the same area numerous precast frame structures collapsed and damage was even found in the steel frame system of an industrial building. Unfortunately, none of the large panel buildings was instrumented and little is known regarding their design strengths and likely level of seismic induced shear force.

There were a number of Bulgarian large panel buildings which were not damaged by the 1977 Vrancea earthquake in Romania,\textsuperscript{9} but the ground motion was reported to have been predominantly long period which would not excite short period shear wall structures. Shapiro\textsuperscript{9} noted that significant

\textsuperscript{*}Harris, J. R., “Precast Building Performance During the Armenian Earthquake,” presented during PCI Committee Days, April 13, 1989.
damage did develop in 2 and 4-story large panel structures during the Gazli 1976 earthquake in the Soviet Union. Some of the vertical panels had residual lateral shifts of 4 to 6 in. (10-15 cm) and floor slabs had slipped off of walls. Damage to panels themselves was negligible. It was postulated that the panel shifting was due to the opening of horizontal joints during the initial motion. Then aftershocks caused gradual progressive displacements. Shapiro also noted that a portion of an actual 9-story building had been tested with shakers at the top to a displacement of 1.8 in. (4.6 cm), causing joint cracking and near doubling of the first natural period.

Other test work has determined the natural periods of in-situ full size buildings through low amplitude shaking with vibration generators. Measured periods of buildings with 4 to 12 stories ranged between 0.17 and 0.52 seconds. Low amplitude tests, however, reflect the natural periods in an "uncracked" state since axial loads generally keep existing cracks closed. Polyakov\(^\text{16}\) noted that large changes in period can develop with large motion and damage. Many tests of joints between panels have been reported, such as those by Hanson, Velkov, Verbic\(^\text{11,12,13}\) and others. Subassembly tests, with statically applied loads, have been completed by Borges, Gavrilovic, and Suenaga.\(^\text{14,15,16}\) Only two reports described dynamic testing of subassemblies; Harris\(^\text{17}\) has tested $\frac{1}{16}$ scale models on a shaking table and Polyakov\(^\text{16}\) noted that vibro-platform tests had been completed in the Soviet Union but he did not provide any measured data. Though there is widespread use of precast large panel construction, there apparently have been virtually no full scale investigations to determine the capacity demands which might be made upon the systems during earthquakes.

**Methods for Predicting Response**

The precast walls act in a manner similar to monolithic shear walls in resisting axial loads and shear forces induced by wind or low amplitude earthquakes. As long as the internal forces do not cause nonlinear response, a precast wall could be analyzed like a monolithic shear wall. When precast large panel wall systems are used in low seismic regions, it may be practical to design them for elastic linear behavior. Elastic behavior may be achieved by ensuring that the forces determined from an elastic analysis, using a response spectrum which has not been reduced, are less than the panel and joint strengths.

Nonlinear response may be created when either the shear force or overturning moment surpasses a limiting value. Horizontal slip starts in the wall joint when the shear becomes too large. The vertical reinforcing steel yields and rocking of the panels starts when the moment becomes too high. Based on the limited existing test data, it appears that slip and flexural distortion are the two mechanisms of damage associated with nonlinear response in a simple wall unit when called upon to resist strong seismic motion. These two types of concentrated deformation are not common in monolithic shear walls because the uniform continuous vertical and horizontal steel tends to distribute the
Several investigators have used the available test data and analytic techniques to attempt to predict and investigate the behavior of wall systems during strong ground shaking. Llorente, and Powell and Schricker proposed analytic means to investigate the sensitivity of panel wall response to various design parameters during strong earthquakes.

Llorente and Becker examined the effects of the postulated rocking motion in a series of analytic studies. They described rocking as an undesirable type of motion since acute shear concentrations at the neutral axis and the severe compressive strains at the closed end of the rocking joint may induce failure and lateral instability. Despite this possible drawback, Llorente's analyses indicated that rocking may be helpful to the wall system. It exhibits an isolation behavior which limits the force which can be transferred into the wall, and softens the wall system, moving the period to a lower point in a response spectra.

Shear slip has been determined experimentally to be a function of the total vertical axial load being transferred through the joint. Since a major portion of the shear may be resisted by pure friction before slip starts, it is likely that slip might occur in joints near midheight of the wall where the total axial force is less than at the base but shear is still high. Llorente investigated the effects of shear slip in analytic studies and concluded that while it represents a source of energy dissipation and force isolation, it should not be counted on as a reliable resistance mechanism because accumulated unrestrained slip could result in enough eccentricity to threaten the stability and integrity of a building.

SHAKING TABLE TESTS

A series of tests was conducted on three large panel wall models as part of this research program to determine what deformation demands and base forces would be developed in such jointed structures under earthquake motion and whether the structure could withstand a strong motion without developing instability. The specific design of connections in the test models was intended to be an improvement on previous forms of jointing for better seismic behavior. The joints were not built in a "platform" fashion as most connections in American systems, but many facets of the response of the tested joints could occur in platform or other types as well. The joints were purposely designed to avoid a shear slip mechanism, for the reasons noted above, while the platform type is expected to fail first in slip. Only a small portion of the description of the tests and deformation mechanisms in the large panel systems can be presented here; a detailed description of the test program may be examined elsewhere.

Test Models

The three large panel test specimens were one-third scale, 3-story high wall segments axially loaded to represent a portion of a wall near midheight of a 15-story building. Each of the specimens was composed of three individual 1-story high wall panels. Every one of the subassemblies was 10.6 in (261 cm) high overall. True scale modeling was employed at a size sufficient to allow use of normal concrete, though with small aggregate. The overall mass of the model had to be artificially enlarged to preserve the correct force ratios. The first model specimen was a simple assemblage of three wall panels. The second model was similar to the first but also included short perpendicular "flange" walls at the ends of the main wall. The third wall included door openings and strengthened lintel beams but it will not be included in the tests described here.
The simple wall had a total vertical reinforcing content across horizontal joints of 0.4 percent and the flanged wall system had 0.7 percent of the wall cross section area. In each case the vertical reinforcement which continued across the joint was concentrated at the extreme ends of the walls as detailed in Fig. 4. The precast panels contained an additional amount of well distributed horizontal and vertical reinforcement throughout their interiors. Each test specimen had a steel platform and a set of mass blocks attached at its top to provide the desired level of internal axial force and to induce lateral inertial forces in the correct scale ratio. The walls were provided with an accessory lateral support frame which allowed vertical and lateral motion parallel to the walls but prevented out-of-plane motion. A test model and support frame are shown in Fig. 5.
Testing System

The tests were performed in the Earthquake Simulator Facility at the University of California Earthquake Engineering Research Center in Richmond, California. The 20 ft (6.1 m) square shaking table was controlled to reproduce the horizontal motion of a recorded earthquake. The wall models were attached to a special foundation which was bolted directly to the shaking table.

Instrumentation was selected to monitor three types of dynamic response: (1) shaking table motion, (2) accelerations and displacements of the models, and (3) local deformations and strains within the models. Horizontal displacements and accelerations were measured at the base, at each floor level, and at the top of each assemblage. Local deformations measured within the test specimens included the shear slip at vertical and horizontal joints, uplift at horizontal joints, and panel shear distortions. Strains were measured in selected reinforcing bars. The special foundation included a set of force transducers to determine the magnitude of base shear transferred into the structure.

Test Program

Each of the specimens was subjected to a series of simulated earthquake motions. The applied motion was proportional to the N-S component of the earthquake recorded at El Centro, California, in May 1940, the intensity being modified by adjustment of the table control system. The El Centro earthquake had considerable energy in the period range near 0.5 seconds characteristic of prototype structures. The recorded earthquake record had to be time scaled to the ratio determined by the true scale modeling of the test specimens. In each case the test signal was first applied at a low intensity to determine the system's elastic response behavior. Then each specimen was subjected to a base motion of sufficient intensity to cause appreciable damage. The test sequence for the two specimens is summarized in Table 1.

The free vibration frequencies of each test structure were measured before and after each simulated earthquake to assess the degree of damage induced during the test. A low intensity "white noise" motion was applied through the table and the acceleration response of the structure was analyzed by Fourier transform procedures.

Damage Observations

Observation of the specimens during and after the tests provides an indication of what damage and inelastic action occurred, supplementing the instrumental data. There was no visible damage during the low intensity shakes. A summary of damage for the intense shakes follows.

Simple wall: The obvious visible response mechanism was rocking motion associated with uplift at the lowest horizontal joint. No shear slip was noted. Uplift was accompanied by apparent compression damage at one end of the wall. Two of the through-joint reinforcing bars at the damaged end of the wall buckled and the third had ruptured. Fig. 6 is a photograph of the damaged area. Only minor cracks existed at the opposite end.

Flanged wall: The visible response was again dominated by rocking associated with alternate uplifting of the wall ends and flange walls. The principal damage consisted of crushing or spalling at one end of the wall and in the adjacent flange; lesser but similar damage occurred at the opposite end. Careful examination of the damaged end revealed that all but one of the five vertical continuous bars in the horizontal connection below the flange and wall had ruptured; the remaining bar was buckled. The opposite end had one flange bar ruptured and two other bars buckled.
Table 1. Test sequence

<table>
<thead>
<tr>
<th>Model</th>
<th>Test No.</th>
<th>Earthquake</th>
<th>Table acceleration (g's)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simple wall</td>
<td>1</td>
<td>El Centro</td>
<td>0.18</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>El Centro</td>
<td>0.67</td>
</tr>
<tr>
<td>Flanged wall</td>
<td>1</td>
<td>El Centro</td>
<td>0.22</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>El Centro</td>
<td>0.69</td>
</tr>
</tbody>
</table>

Table 2. Measured natural frequencies.

<table>
<thead>
<tr>
<th>Model</th>
<th>Time of measurement</th>
<th>Frequency (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simple wall</td>
<td>Prior to test</td>
<td>5.20</td>
</tr>
<tr>
<td></td>
<td>After Test 1</td>
<td>5.10</td>
</tr>
<tr>
<td></td>
<td>After Test 2</td>
<td>3.90</td>
</tr>
<tr>
<td>Flanged wall</td>
<td>Prior to test</td>
<td>6.30</td>
</tr>
<tr>
<td></td>
<td>After Test 2</td>
<td>4.32</td>
</tr>
</tbody>
</table>

Test Results

The reduction in frequency (Table 2) shows that the stiffness decreased by nearly 50 percent as a result of the damage during the strong tests and verifies concerns noted by Polyakov and Hawkins\textsuperscript{10,23} regarding the large period changes which can occur in panel wall systems. Hawkins suggested that the ground motion may not be significantly amplified in panel structures if their period remains at 0.3 seconds or less (a period of 0.16 seconds for a one-third scale model, slightly lower than the actual measured period). Softening can lengthen the period if joint slip or rocking starts and could result in the structure showing major amplification of the ground motion.

The instrumentation confirmed the predominance of the rocking response described above during the strong shakes. The structures remained elastic during the low intensity tests. Peak deformation quantities in Table 3 indi-

![Fig. 6. Close-up view of damage at the end of the simple wall; the bar which is not buckled had ruptured.](image-url)
Table 3. Peak deformation quantities.

<table>
<thead>
<tr>
<th>Loading conditions</th>
<th>Simple wall</th>
<th>Flanged wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test 1</td>
<td>Test 2</td>
</tr>
<tr>
<td>Table acceleration (g's)</td>
<td>0.18</td>
<td>0.67</td>
</tr>
<tr>
<td>Acceleration at top of wall (g's)</td>
<td>0.28</td>
<td>0.80</td>
</tr>
<tr>
<td>Displacement at top of wall (in.)</td>
<td>0.13</td>
<td>1.48</td>
</tr>
<tr>
<td>Uplift at end of wall (in.)</td>
<td>0.02</td>
<td>0.70</td>
</tr>
<tr>
<td>Base shear (kips)</td>
<td>7.4</td>
<td>15.7</td>
</tr>
<tr>
<td>Base moment (in.-kips)</td>
<td>1525</td>
<td>3516</td>
</tr>
</tbody>
</table>

cate that an amplification of the ground motion occurred in the structure. The rocking response during the strong motion tests was indicated by the uplift gages at the base of the wall across the lowest joint. The nature of the uplift may be deduced from Fig. 7 which shows the measurement at three points on one side of the simple wall. Gage U6 is located at the extreme end; it clearly shows that the wall uplifts a maximum of 0.34 in. (0.86 cm) as the top displaces sideways in one direction. It is also clear that the uplift never completely returns to zero. The displacement at Gage U8 shows similar but less uplift in synchronization with Gage U6 indicating that the point of rotation is near the opposite end of the wall. The smaller peaks, which are out of synchronization with Gage U6, occur when the opposite end of the wall uplifts to a maximum of 0.70 in. (1.8 cm).

The second type of primary motion measured at the lower joint was slip, i.e., sliding of the two vertical walls relative to each other. The average sliding motion, though only reaching an amplitude of 0.04 in. (0.1 cm), tends to be toward the south — the end which experienced concrete damage and spalling.

A final indication of the performance of the simple wall may be obtained from an examination of its base shear vs. top displacement history as shown in Fig. 8. Overturning moment controlled the rocking response, but the shear was linearly related to the moment. The moment-shear ratio was 204 in. or 2.7 times the length of the wall. First yield in the joint steel was measured at a moment of 1790 in.-kips (202 kN-m). The calculated yield strength of the wall was estimated as 2070 in.-kips (234 kN-m) or with a shear of 10.1 kips (45 kN) using normal beam theory. The difference in predicted and actual first yield was a result of the plane section assumption used in normal beam theory. The behavior of the wall was essentially linear until the shear exceeded –12.8 kips (–57 kN). On reversal, the stiffness decreased and further damage occurred at +15 kips (67 kN). The sudden drop in shear corresponded with rupture of one of the joint bars. On subsequent cycles the positive shear never reached the same maximum value.

Similar rocking and slip response was measured in the tests of the flanged wall though its stiffness and strength were obviously higher than in the simple wall. A very small amount of deformation occurred in the vertical joint between the main wall and the flanges but did not dissipate significant energy. A full description of the test results and data measurements is given in Ref. 24.
Fig. 7. Uplift histories of the panel at the lower connection.
Extensive computer aided studies have been completed at the University of Wisconsin using data from the shaking table tests to verify the ability of an existing analytic method. The experimental vs. analytic correlation studies identified the essential response mechanisms which had to be duplicated for successful prediction of the seismic response of the precast panel wall system at various levels of excitation.

**Analytic Modeling**

The primary portion of the analytic model consisted of the three stories of precast walls and joints. A lumped translational and rotational mass was placed above the wall elements and attached to the wall with rigid links to simulate the mass blocks above the test specimens. The wall elements were connected to a spring foundation which modeled the flexibility of the actual foundation member and force transducers. Finally, the shaking table itself exhibited a detectable amount of pitch during each test (due to structural interaction) even though the command motion specified only a horizontal component. The analytic model included a rigid table with rotational mass supported on springs to simulate the dynamic table pitching. This entire model is diagrammed in Fig. 9.

Certain assumptions were initially made during the modeling: (1) the precast panel elements were assumed to maintain a linear stiffness and remain elastic; (2) all inelasticity was assumed to be concentrated within the connection regions, and (3) the connection regions were assumed to be precracked due to likely shrinkage between the joint grout and the precast panels.

**Techniques for Joint Modeling**

Horizontal joints between vertical precast panels were assumed to be the locations where all inelasticity would take place and the analytic joint elements had to represent the inelastic mechanisms likely to develop. Insufficient information exists at present to model the entire range of possible joint behavior with a single analytic “connection element.” Two approaches have been used to model this connection re-
Fig. 9. Diagram of the 3-story analytic model.
gion: (1) as a continuous media, with multinode inelastic rectangular contact or interface finite elements, or (2) with discrete inelastic nondimensional spring elements. Becker, Mueller and Llorente discuss the advantages of using interface or contact finite elements in obtaining gradual opening of the joint and avoiding impact problems and the required discretization if spring elements are used.

Regardless of the modeling approach, the simulation of the joint concrete may need inclusion of initial compressive stiffness, crushing with stiffness and strength degradation, and opening of a gap when under tension. The concrete is represented with finite elements or a series of discrete springs. The vertical tension reinforcement is usually explicitly modeled with truss elements. Shear slip resistance may be included in the continuous concrete finite elements or by special nondimensional slip springs.

The use of discrete springs in the form suggested by Powell and Schricker was applied in this study with a modified version of the DRAIN2D-MkII computer program. Use of discrete nondimensional springs makes it easier to develop new elements to model specific characteristics.

Our test results clearly indicated that successful modeling would require duplication of the rocking mechanism with attendant gap opening and closing. Moreover, it appeared as if the model should be able to reproduce rupture of the steel reinforcement across the horizontal joint since rupture occurred very quickly once the bars started yielding. As a consequence of these two requirements, particular attention had to be given to the form of the timewise analysis algorithm since each of the preceding events would abruptly alter the stiffness of the system. If a constant time step method were used, small time increments would be necessary, resulting in significant computation time. In other joint configurations, without large shear keys, shear slip may be more significant and require special simulation.

**ANALYTIC CORRELATION RESULTS**

**Elastic Behavior**

The earthquake motion reached a peak acceleration of 0.17g during the low amplitude test of the simple wall; this level of motion was not sufficient to cause visible cracking or yielding. Excellent correlation was achieved between the measured experimental and predicted top displacements throughout the time history when the correct section properties were used in the analytic model. To obtain “correct” section properties, the lower panel’s effective cross section had to be able to change from gross to cracked when moments exceeded the cracking level. It was then necessary for the model to regain its gross section stiffness as a condition for matching the low amplitude response cycles after cracking had occurred. The axial gravity load causes the wall to redevelop its gross section when the overturning moment is low.

**Inelastic Behavior**

Simulation of the simple wall’s inelastic response during strong ground motion required an exceptionally complex modeling approach. The system’s response involved cracking of panels, yielding of steel in the joint, gap opening, buckling of steel rods, rupturing of steel, and spalling and destruction of unconfined concrete near the buckled vertical bars. The effect of these various mechanisms in modifying the predicted response of the wall system is discussed in this section.

The simplest inelastic model used bilinear yielding truss elements to represent the joint’s steel reinforcement. The predicted response to the 0.67g earth-
Fig. 10. Response history of the analytic model without rupture capability (rupture occurs at 1.42 seconds in the test specimen).

Fig. 11. Response history of the analytic model which included rupture (rupture occurs at 1.42 seconds).

The inclusion of steel rupture was essential for accurate modeling of the true behavior since the predominant rocking mechanism became active after rupture. Fig. 11 shows the change in simulated response which occurred when the truss

quake using a time step of 0.001 seconds is compared to measured response in Fig. 10. The simulation gave good correlation until the joint reinforcement ruptured in the test specimen at 1.42 seconds.
element had a limiting value at which rupture occurred. The predicted base shear vs. top displacement plot, which is shown in Fig. 12, produces the same nonlinear elastic rocking response after rupture as was seen in the experimental data of Fig. 8.

Many additional effects had to be considered to achieve a satisfactory simulation of the precast system's inelastic response. For instance, the wall panels do not bend in a manner consistent with the simplified theory of "plane sections remain plane." The gap at the tension end, during rocking, becomes larger than expected and the compression zone at the opposite end becomes smaller than expected. Reliable modeling of the joint resistance requires use of numerous closely spaced discrete concrete compression gap elements at the ends of the wall due to the tremendous strain variation in a short distance. Malhas\(^{25}\) has listed items which had to be specifically simulated or considered in developing the analytic model with the response shown in Fig. 10 including:

1. Nonlinear strain variation over a cross section.
2. Yielding across the thickness of the joint or over the unbonded length for reinforcement which is made continuous across the joint.
3. Rupture of steel crossing a joint.
4. Buckling of unruptured steel after it has undergone tension stretching and is then recompressed.
5. Concrete cracking and gap opening.
7. Decrease in concrete compression stiffness after a gap opening has occurred.
8. Simulation of shear slip mechanisms if necessary.
9. Inclusion of closely spaced nodes at the interface between panels and joint elements to avoid force concentration at particular points in the panel and to simulate the rapidly varying joint forces caused by the nonlinear strain variation.
10. Use of small time steps or variable time stepping to avoid integration errors when stiffnesses abruptly change.

Fig. 12. History of the base shear and top displacement as predicted by the analytic model.
RESPONSE PREDICTION FOR 13-STORY PROTOTYPE

Having achieved successful correlation between the measured response and predictions for the test model, the analytic techniques were employed in estimating strong motion response and defining certain limit states for the full 13-story prototype wall system upon which the model sections in the experimental work had been based. Panel and joint stiffnesses were assigned on the basis of experience with the one-third scale model. Because of the height of the prototype system, the initial elastic first mode natural period was nearly 0.8 seconds.

The limiting earthquake level for the wall system to avoid yielding was found to be at a maximum acceleration of 0.36g if the ground motion was proportional to the El Centro motion used in the wall test program. The peak acceleration would vary considerably for other types of motion since the yielding level was found to be very sensitive to the match between structural natural frequency and the earthquake spectra. A second limiting level of ground motion, again for a motion proportional to the El Centro record, would be the amplitude which would cause rupture of the vertical reinforcement. The predicted prototype rupture would occur when the ground motion reached an acceleration amplitude of 0.9g though concrete crushing initiates in the joint at an amplitude of 0.5g and may cause deterioration of the system's stability before rupture could occur.

SUMMARY AND DISCUSSION

Project Description

Three precast panel wall subassemblies were tested under simulated earthquake motion. Each of the specimens was a 3-story one-third scale model of walls from near midheight of a high rise building. The individual precast panels had very limited vertical reinforcement (0.4 to 0.7 percent) made continuous across joints at each floor level. The reinforcement was concentrated at the ends of the wall with a prototype spacing of 206 in. (5.23 m). The models experienced shaking from ground motion proportional to the El Centro earthquake with their response measured and compared to analytically predicted behavior.

Base Shear During Earthquake Testing

The 3-story simple precast wall subassembly examined here was designed to elastically resist a base shear equal to 45 percent of the system's weight. If designed by the Uniform Building Code (UBC), the wall would be required to resist a base shear of approximately 20 percent of the system's weight for construction in a high seismic area (Zone 4). Thus, the test subassembly was designed to resist a force of just over twice the UBC's required minimum. A moderate seismic motion with an acceleration amplitude of 0.2g was successfully resisted by the system without perceptible damage or measured yielding while peak base shears as high as 32 percent of the system's weight were developed. When the ground motion was increased by a factor of 3 to 4, to a peak acceleration of 0.7g, significant observable damage developed with yielding, uplifting, and rocking at the lower horizontal joint. At the higher level of motion a base shear equal to 70 percent of the weight was developed, indicating that the force demand created by the earthquake was approximately equal to the product of ground acceleration and system mass. The base shear reached 150 percent of the design base shear.

The base shear which develops during seismic shaking of a structure depends on the dynamic nature of the structure, specifically its natural periods...
and damping. Thus, the characteristics a
designer gives to a structure can influ-
ence the base shear or strength demand
and displacements which an earthquake
will create. In the test structure the de-
sign shear capacity at yield was less than
the earthquake induced force and
yielding ensued. The yielding resulted
in a 50 percent reduction of the system's
stiffness causing a significant change in
the natural period. The yielding and
change of period has a direct influence
on what level of force demand the
earthquake will tend to create within
the structure.

Structural Capacity Demand and
Design Strength
Minimum design forces such as
specified in UBC are lower than the ac-
tual forces which would be induced in a
structure by a design earthquake if the
structure were to remain elastic. The
codes allow design for reduced forces
under the assumption that during a de-
sign earthquake the structure will yield
when the reduced design force level is
reached and plastic deformation will
take place until the reversing nature of
the earthquake causes the forces to de-
crease. It is implicitly assumed that the
structure will be able to withstand this
yielding and plastic deformation with-
out failing. Unfortunately, our under-
standing of the dynamic mechanisms
and available plastic deformation in
precast concrete structures is very in-
complete, making it difficult to take ad-
vantage of reduced force design.

The connection below the lowest pre-
cast panel was forced to sustain large
deformations in the wall system tested.
The connection yielded because the
base shear which the earthquake motion
would have created, if the structure had
remained elastic, was more than three
times the yield strength of the connec-
tion. Since the connection was not able
to remain elastic, it was forced to
undergo plastic deformation with dam-
age. The amount of top displacement
which accompanied the extensive
yielding was dependent on the design
yield force level and reached more than
five times the yield displacement before
the strength capacity became reduced
(displacement ductility = 5). In fact, the
maximum top displacements, with
rocking after joint reinforcement had
broken, became as high as ten times the
yield displacement. Seventy-five per-
cent of the top displacement was due to
the rocking of the bottom joint.

Connection Capacity
Demand — Rocking Motion
A rocking mechanism effectively iso-
lated the test walls from the ground mo-
tion and limited the amplitude of base
shear which could be transferred into
the structures. During rocking the sys-
tem's weight, acting along a path near
the center of the wall, provided the only
restoring moment to counteract the
overturning moment caused by inertial
effects. This constant resistance capacity
is clearly evident in the negative dis-
placement cycles of Fig. 8. This mech-
anism isolated the wall above the joint
from receiving any greater moment or
shear. The load limiting effect, initiated
by opening of the lower joint, however,
prevented the spread of inelasticity to
any other locations in the wall system.
The reinforcement which was continu-
ous across the lower horizontal joint ex-
perienced tremendous elongation and
rupture since nearly all the deformation
demand created by the ground motion
was concentrated within the single joint.
Strains in the limited cross joint rein-
forcing bars surpassed 4 percent, the
level at which strain gages became de-
fective. The average bar strain calcu-
lated from the joint uplift if the bar had
not ruptured would have been as high as
32 percent, far beyond the bar's strain
capacity.

A maximum uplift of 0.7 in. (1.78 cm)
occurred in the model test wall. The
equivalent uplift in the 13-story prototype building would have been 2.1 in. (5.3 cm). Under these conditions the closed end of the joint is under tremendous compressive stress since the axial load and flexural compression is resisted within a very small concentrated compression zone. This zone must be able to resist high compression forces without brittle crushing. The compression zones in the test specimens exhibited limited crushing of the joint concrete. The crushing naturally started at the outer fiber and proceeded inward as material was lost. Crushing only occurred over a limited distance near the ends of the walls in the test specimens. The amount of crushing which occurred appeared to be limited by the short length of time within which the wall was at a high uplift. Reversal of the uplift, caused by the reversing ground motion, reduced the compression force and limited the degree of concrete crushing.

Connections play the most important role in controlling the behavior of a precast large panel wall system during seismic loading regardless of the design approach used. Though current practice in aseismic design is aimed at developing strong connections and forcing inelastic behavior away from connections, just the opposite behavior occurs in precast systems. Weak connections can, however, operate successfully if their design explicitly provides for the inelastic demands of earthquake motion. The level of yield strength has to be balanced with sufficient deformability and failure strength to allow repeated cycles of shaking without collapse.

The connections between panels of the wall system described in this paper were designed to yield and fail in flexure before shear. Large shear keys prevented a premature shear failure. Panel wall systems using the platform connection of the United States are likely to fail first in shear. Llorente and Becker's investigations of the benefits and disadvantages inherent in flexural or rocking type failures and shear failures showed that both types of failure could lead to instability.

The shear failure mechanism (shear slip) is capable of dissipating energy very efficiently so that the energy transferred into the structure by the ground motion does not create large forces or large displacements. Llorente found nevertheless that shear slip may be undesirable because resistance depends largely on friction and when sliding starts it is liable to lead to accumulated unrestrained displacements under certain earthquakes when the slip occurs predominantly in one direction. There is certain danger in having unrestrained displacement because large secondary (P-Delta) moments develop and eccentricity will occur in perpendicular walls.

A rocking mechanism dissipates little energy as could be seen in Fig. 8, and creates severe force concentration in the compression region. As the wall rocks open, all the axial load and the compression force of the flexural couple has to be resisted in a small compression stress zone at one end of the wall. Compression crushing of the concrete may occur, leading to instability. Rocking's main advantage is that it should not result in accumulated displacements.

The importance of connection design in precast large panel wall structures with the two mechanism alternatives, decisions regarding design strength and associated deformation demand, and limited knowledge of available ductility leave a designer in a quandary when attempting to provide an efficient and safe system. Both of the mechanisms noted above create softening of the structural system with increasing displacement and may act to isolate the remainder of the wall from increased force transfer; however, they also prevent the spread of inelasticity in the wall. Overall it appears that the flexural or rocking type of mechanism is preferable in its resistance to developing accumulated
displacements. If the flexural mechanism is selected, then it remains for the designer to insure that the joint has a sufficient balance of strength and ductility or deformability to survive the seismic demands. The best balance of those quantities has not been determined but the model test walls have sufficient capacities to withstand the effects of a major seismic motion.

**Toughness of Large Panel System**

The ability to survive the effects of an earthquake, through strength, energy dissipation, ductility and deformability, has often been referred to as toughness. Though the walls examined in this study exhibited little energy dissipation and only moderate ductility in their rocking mechanism, they did endure a strong earthquake test. The combination of force isolation, varying stiffness and period, and ability of the system to undergo large displacements associated with rocking while maintaining its stability allowed it to maintain vertical load carrying without collapse. The toughness exhibited in the walls appears to be primarily a result of the deformability and the force isolation effect.

The large panel building system can also be designed to withstand a major earthquake by providing toughness in the form of a large elastic energy absorption capacity. The precast panel wall buildings which survived the December 1988 Armenian earthquake were very similar in construction to the walls described here and appeared to have been provided with such a capacity. The very nature of the application of many large panel systems, to provide housing, tends to make it easier to achieve high elastic strength capacities. In apartment buildings many of the interior walls can be load resisting panel walls. With numerous walls available, the lateral load capacity may become quite high. The horizontal joints between panels need reinforcement which is continuous between panels, but a small amount of reinforcement which is connected at the end of each wall can provide a considerable moment resisting capacity due to its large moment arm combined with the axial compressive stresses in the wall from gravity loading.

**Evaluation of Analytic Techniques**

A set of criteria which needs to be included in an inelastic analysis to correctly simulate response has been determined for wall rocking mechanisms. Particular attention must be given to the special conditions which exist during the rocking type of response: nonlinear cross section, opening and closing of the joint gap, rapid changes in stiffness, and degrading material characteristics. New elements had to be added to an existing panel wall analysis program to simulate all the required joint stiffness characteristics. The integration time step in a step-by-step analysis had to be carefully chosen to avoid errors caused by abruptly changing stiffness. Beam modeling of the cantilever wall system was not successful once the joint crack opening reached a stage where nonlinear strains exist across the section. Even in the elastic range (before yielding) the wall’s response is particularly sensitive to changes between gross and cracked section stiffnesses.
CONCLUSIONS AND RECOMMENDATIONS

Based on the three specimens tested in this program and the corresponding analytic simulations, certain conclusions can be drawn.

1. Shear walls of large panel precast construction can be designed to resist loads induced by moderate earthquakes while remaining elastic and being easily constructed. Even though the systems have weak connection regions, with only a small amount of continuous reinforcement through the joints, significant capacity can be achieved. When the design is based on full elastic loads, it is not necessary to provide connections which resemble monolithic concrete.

2. When shear wall systems, and particularly the joints, are designed to resist forces which are lower than the likely elastic force which would be induced by the design earthquake, such as design forces which are often given in codes, inelastic action will likely result during a design earthquake. If the construction is similar to the test specimens, then the joints may undergo extreme deformations, such as the rocking noted in the tests, and joint reinforcement may rupture, but there is a good likelihood that the system will be able to survive the ground motion without collapse. A rocking mechanism will result and would need special provisions to maintain stability.

3. The particular large panel system tested in this program appears to have the potential to be able to survive strong seismic motions without collapse though experiencing serious deformation and damage.

4. Energy dissipation, which is very low with a rocking mechanism, may not need to be a prime objective in the design of the large panel precast seismic resistant systems if sufficient deformability is provided while maintaining stability. Minimum ties as suggested by PCA are essential to maintaining stability in a three-dimensional structure and must be provided when inelastic behavior is anticipated.

5. Analytic techniques exist which are capable of predicting the behavior of large panel wall systems but are unpractical or unreliable for normal use because modeling is complex and the response of the system is very sensitive to the changing stiffness of the connection region.

The design of precast structures to have acceptable seismic resistance is a perplexing problem at this time because of their weak jointed nature. Recent changes in design codes will require buildings to be designed for seismic load in portions of the United States where wind loading once controlled. Additionally, the demand for economical buildings, particularly for housing, has created a need for means of seismic resistant design in regions of strong motion. Certain steps may be taken to treat the current design problems in regions of low seismicity and to develop designs in the future for regions of high seismicity.

A. Large panel buildings should be designed to elastically resist the seismic forces in regions of low seismicity. Current technology allows such design when seismic forces are not high. When codified equivalent static loading methods are used, the design base shear should not be a “reduced” force. The ATC approach for defining the base shear could be employed without the load reduction factor “R”. Since it is often possible to use many walls in the building as lateral load resisting structural walls, the actual force developed in each will often be relatively small as in the panel wall buildings surviving the Armenian earthquake.

B. The rocking mechanism, which occurs when the elastic capacity is passed, appears to be the more desirable of the
two likely inelastic mechanisms possible in a horizontal joint between wall panels (i.e., shear slip or rocking). The rocking mechanism is unlikely to develop unrestrained motion or accumulated deformation which could lead to instability.

It appears that it would be desirable to modify the platform system of construction used in the United States in a manner which would limit slip and create flexural motion. This might be achieved by providing grouted keys or shear resisting links between the stacked wall elements. If the platform system can be modified to force the inelastic mechanism to become a flexural one, then steps must also be taken to provide an increase in the compressive strength in the joint region. The platform system would probably not have sufficient compression capacity to resist the necessary loads in a ductile manner and development of a modified joint would be necessary.

C. Particular attention must be given to tying the entire building system together as a means of preventing accumulated deformations from developing and for maintaining stability. As a minimum, the PCA recommendations for ties around the periphery and through the diaphragms of the system must be provided.

D. Attention should be aimed at the use of vertical connections between stacks of panels as the first location of inelasticity rather than rocking or slip in the horizontal joints between panels. Vertical joints could be used to form a coupled shear wall system analogous to the system recently developed for monolithic construction. A limited amount of research has been dedicated toward this end but apparently has not been successful yet.

A mechanism within the vertical joint would solve three of the basic problems in inelastic panel wall response. First, since the vertical connection would not be an essential link in the gravity load bearing system, its loss would not endanger the stability of the building system. Secondly, the joint could be a source of energy dissipation since it would serve primarily as a shear transfer mechanism. This would complement the lack of energy dissipation and ductility of the existing rocking mechanism. Third, having a vertical coupling joint between walls would create redundancy in the wall system. It is well recognized that redundancy is a very desirable feature in any structural system which may be loaded beyond its elastic limit.

E. Large scale tests and analytical simulations, except for the Armenian earthquake results, have involved only two-dimensional assemblages. It is unclear how the complete three-dimensional building system will act. Either the slip or rocking mechanism, occurring in walls in one direction, will affect the strength and stability of other walls running in a perpendicular direction, since those walls will presumably be bent about their weak axis. Analytical approaches will have to be developed to model the three-dimensional behavior and tests should be used to verify predictions since stability effects are difficult to simulate in an inelastic system.

F. Large deformation in the horizontal joints between vertical panels, either slip or particularly rocking, will exert serious deformation demands upon the floor or roof diaphragms. Rocking in a three-dimensional structure may literally tear the floor diaphragms apart. Special reinforcement and ties may be necessary to ensure the integrity of such diaphragms but insufficient information is available at present to provide any design guidelines. The bending deformability of precast floor diaphragms should be substantiated.

* * *
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REFERENCES

Portugal, 1966.


NOTE: Discussion of this paper is invited. Please submit your comments to PCI Headquarters by June 1, 1990.