The effectiveness of using friction type mechanical connectors along vertical joints to improve the earthquake resistance of large panel precast systems is investigated. In particular, the paper examines the relative roles of these connectors and gap-friction action in the horizontal joints. Nonlinear time history dynamic analysis is employed to determine the design connector slip load at which response is optimized. The results show that reduced seismic response and enhanced structural integrity are achieved through use of such connectors. Design implications, as well as a tentative procedure for selecting these devices, are also discussed.

Earthquake ground motions cause structures to respond in proportion to the amount of the seismic input energy. Efficient control and dissipation of this energy is necessary to ensure that buildings escape serious damage. In modern philosophy of earthquake resistance, ductility has been accepted as the prime design criterion. However, the limited inherent ductility of precast concrete large panel (LP) structures has led to doubts concerning their capacity for energy absorption. Furthermore, the generally higher stiffness associated with such buildings over conventional framed systems invites higher inertial forces, thereby placing greater demands on strength and ductility.

These concerns, coupled with a general lack of data on the seismic response of large panel buildings, have contributed to limiting the use of LP construction in the seismically active zones of North America. Indeed, the widespread destruction of precast concrete buildings during the 1988 earthquake in Armenia provides dramatic evidence not only for the need to ensure adequate design and good quality in construction but also for the additional need to examine possible means to improve the seismic resistance of such structures.

In 1980, Pul1 et al. introduced the concept of friction joints in order to minimize damage in precast panel structures. They performed analytical studies on the seismic response of
coupled LP shear walls to determine the effectiveness of employing limited slip bolted (LSB) friction type mechanical connectors located along the vertical joints. It was demonstrated that walls coupled through such LSB connectors could be tuned to provide optimum response by varying the connector shear slip load. Thus, the LSB connectors were shown to act both as safety valves in releasing load on the vertical connections and as dampers to reduce the overall response of the structure.

The study assumed that the precast walls of the LP system act as two coupled continuous elastic cantilevers; hence, no horizontal joint action was considered and nonlinear behavior in the structure was confined to the vertical joint only. Full vertical continuity was said to exist either because of post-tensioning or due to large gravity load sufficient to prevent shear slip and gap opening in the horizontal joints.

Shear slip and rocking, associated with such horizontal joint action, can affect the behavior of a precast panel system considerably. The analytical studies by Becker and Llorente, Shricker and Powell, and Kianoush and Scanlon all revealed the influence of horizontal joint action on seismic response, while experimental work conducted by Oliva and Shahrooz as well as Harris and Caccese confirmed the importance of the rocking and shear slip mechanisms related to the horizontal joints.

In an attempt to develop a more general approach toward the anticipated behavior of precast shear walls equipped with LSB connectors, the present study incorporates the existence of horizontal joints. Rather than modeling the precast shear wall as only two continuous cantilever walls coupled along the vertical joint by LSB connectors, wet platform type horizontal joints common of North American LP practice are introduced and assigned properties based on available experimental data.

In place of the simplified wide column frame analogy of Reference 1, a finite element idealization of the structure is employed. The latter comprises: (1) panel elements possessing linear elastic behavior; (2) horizontal joint elements exhibiting gap opening and shear slip; and (3) limited slip bolted (LSB) connectors along the vertical joint acting primarily in shear.

**PROTOTYPE STRUCTURE**

The prototype wall chosen for this study consists of one of the end walls of a typical 10-story precast LP building of the crosswall type, for which the plan layout is shown in Fig. 1(a). Compared to the interior walls, the end walls possess greater lateral stiffness; thus, they attract more severe lateral forces during seismic activity while, at the same time, carrying less...
gravity load. These considerations make the end walls more heavily dependent on the gap friction action of the horizontal joints. In addition, whereas the two halves of each end wall are coupled along the vertical joints, the weaker shear slip are assumed to be prevented. In practice, this will require the same vertical reinforcement as for conventional design of monolithically constructed walls or, alternatively, bolted horizontal joint connectors of equal capacity. Although proposed in Reference 1, additional numerical results not reported herein indicate that gravity load or post-tensioning appear unable to eliminate horizontal joint action.

For precast walls allowing nonlinear behavior of the joints, the weaker platform, or “American,” Type B horizontal joints illustrated in Fig. 2(b) are modeled; a compressive strength of 14.5 MPa (2.1 ksi) and elastic modulus of 13,800 MPa (2 x 105 ksi) are assumed, based on CPCI’ design formulas and data as well as on the results of tests of full scale horizontal joints by Harris and Iyengar. Vertical mild steel reinforcement through the horizontal joints equal to 0.5 percent of the gross cross-sectional area of the joint is provided in the Type B1 version of the prototype LP wall, whereas the Type B2 wall is assumed to be unreinforced.

The vertical joint between wall panels comprises LSB mechanical connectors, provided two per panel at the edges. Fig. 3 illustrates the basic components of these connectors, which are described in greater detail in Reference 1. Each connector consists of a steel plate with slotted holes that is bolted on site to inserts anchored into the concrete panels. The slotted bolt holes allow for normal fabrication and erection tolerances as well as clearance for the expected slip. Friction pads are inserted between the steel connecting plates and the inserts to produce cyclically stable elasto-plastic behavior of the connection. Two M22 (7/8 in.) A325 bolts and two 155 x 19.1 mm (6½ x ¾ in.) headed stud anchors are employed per side for the prototype LSB connectors of the Type B structures.

Depending on the level of prestress in the bolts, the LSB connectors allow for a variable shear slip resistance, the so-called slip load \( F_{s,0} \), and an ultimate shear capacity of 254 kN (57 kips) if bearing occurs following slip. Although the anchorage is assumed to be elastic, its computed yield strength is 160 kN (36 kips) and thus, while yielding of the anchor studs is avoided at the anticipated optimum slip load \( F_{s,0} = 80 \text{ kN (18 kips)} \), anchorage yielding would occur prior to shear failure of the bolts. The pull-out capacity of the stud anchors is 325 kN (73 kips); however, the tensile capacity of the connector is governed by the shear capacity of the bolts and is thus equal to 254 kN (57 kips) also.

As Fig. 3 demonstrates, the LSB connector allows slip to occur only in the vertical direction, with rotation prevented by welding one side of the plate to the insert.

### Table 1. Properties of prototype wall.

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Panels and loading</td>
<td></td>
</tr>
<tr>
<td>Panel thickness (mm)</td>
<td>200</td>
</tr>
<tr>
<td>Modulus of elasticity (MPa)</td>
<td>( 29.2 \times 10^3 )</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.15</td>
</tr>
<tr>
<td>Compressive strength (MPa)</td>
<td>34</td>
</tr>
<tr>
<td>Gravity load/story (kN)</td>
<td>490</td>
</tr>
<tr>
<td>Tributary story mass (kg)</td>
<td>( 128 \times 10^3 )</td>
</tr>
<tr>
<td>(b) Type B1 horizontal joints</td>
<td></td>
</tr>
<tr>
<td>Thickness (mm)</td>
<td>200</td>
</tr>
<tr>
<td>Modulus of elasticity (MPa)</td>
<td>( 13.8 \times 10^3 )</td>
</tr>
<tr>
<td>Compressive strength (MPa)</td>
<td>14.5</td>
</tr>
<tr>
<td>Axial parameters: ( k_1 )</td>
<td>13.15</td>
</tr>
<tr>
<td>( k_2 )</td>
<td>2.39</td>
</tr>
<tr>
<td>( k_3 )</td>
<td>0.001</td>
</tr>
<tr>
<td>( k_4 )</td>
<td>0.25</td>
</tr>
<tr>
<td>( u_1 ) (mm)</td>
<td>0.176</td>
</tr>
<tr>
<td>( u_2 ) (mm)</td>
<td>0.420</td>
</tr>
<tr>
<td>Shear parameters: ( k_5 )</td>
<td>5.75</td>
</tr>
<tr>
<td>( k_6 )</td>
<td>0.575</td>
</tr>
<tr>
<td>( r )</td>
<td>0.15</td>
</tr>
<tr>
<td>( \mu_0 )</td>
<td>0.40</td>
</tr>
<tr>
<td>(c) Vertical joint LSB connec ters</td>
<td></td>
</tr>
<tr>
<td>Axial stiffnesses: ( k_a )</td>
<td>1950</td>
</tr>
<tr>
<td>( k_v )</td>
<td>490</td>
</tr>
<tr>
<td>Shear stiffness: ( k_s )</td>
<td>640</td>
</tr>
<tr>
<td>( k_0 )</td>
<td>320</td>
</tr>
</tbody>
</table>

Note: See figures of Appendix A for definitions of parameters related to joint behavior.

Metric (SI) conversion factors: 1 mm = 0.0394 in.; 1 MPa = 0.145 ksi; 1 kN = 0.225 kip; 1 kg = 2.20 lbs.; 1 kN/mm = 5.71 kips/in.; 1 kN/mm/mm = 145 kips/in./in.
STRUCTURAL IDEALIZATION

The analysis is accomplished through a finite element representation of the coupled wall structure. The earlier study employed a simplified approach by idealizing the coupled walls as an equivalent wide column frame, connected by modified truss elements to represent the LSB connectors. However, with the introduction of horizontal joint action, the wide column analogy is no longer valid.

Instead, the panels are modeled as an assemblage of linear elastic plane stress finite elements and discrete two-node, four degree of freedom, orthogonal spring elements are used to model both the continuous horizontal joints and the LSB vertical joint connectors. The elements are defined across the interpanel joints using the plane stress element nodes and possess properties in both the normal and shear directions.

Nonlinear inelastic action and coupled shear-axial behavior is possible. For the Type A horizontal joints, rigid material behavior is assumed for the joint elements in both the normal and shear directions. A single spring element suffices to model the behavior of each LSB connector, whereas five such elements are used to approximate the behavior of each continuous horizontal joint between panels.

IDEALIZED BEHAVIOR OF JOINTS

The force-deformation relationships employed to model the expected behavior of the joints are provided in Appendix A, while Tables 1(b) and 1(c) contain the magnitudes assumed for the various parameters involved.

Horizontal Joints

Axial behavior normal to the hori-
Horizontal joints\textsuperscript{11-15} follows a trilinear curve in compression, with crushing assumed to commence at the joint compressive strength $f'_c = 14.5$ MPa (2.1 ksi). Tension accompanying gap opening is assumed to be governed by elastic behavior of the vertical reinforcing steel. On the other hand, behavior in shear is based on the shear-friction model employed by Kianoush and Scanlon.\textsuperscript{16-17} In this model, dowel action due to the vertical reinforcement restores shear resistance once the available friction sliding strength is reached.

**Vertical Joints**

Tension-compression forces across these joints are assumed to be transmitted based on corresponding elastic stiffness of the LSB connectors. In resisting shear along the joint, the LSB connectors slip (i.e., exhibit stable elasto-plastic behavior\textsuperscript{1}) when the slip load $F_{sb}$ is reached, prior to which the load-deformation relationship is elastic. During slip, the bolts move within the slotted holes. When the end of the slot is reached the bolts bear against the plate and elastic behavior is restored. In the analysis, however, the possibility of bearing of the bolts is reserved for comparison purposes only and slot lengths are otherwise assumed to be sufficient to allow unrestricted bolt slip. Anchorage into the panel concrete is assumed to remain elastic.

**DYNAMIC ANALYSIS**

The study was accomplished using an expanded version of the Fortran program ANSR-I (Analysis of Nonlinear Structural Response),\textsuperscript{18} implemented on a SUN microcomputer system. Viscous damping corresponding to 5 percent in the first and second modes was assumed. An integration time step of 0.001 seconds was used to obtain the time-history response employing the first 6 seconds of the north-south component of the 1940 El Centro earthquake record. This was followed by 1 second of zero ground acceleration, for a total of 7 seconds of analysis.

**RESULTS AND DISCUSSION**

Seismic response in the prototype wall is examined assuming either strong (rigid) Type A or nonlinear behaving Type B horizontal joints, the latter allowing both gap opening and shear slip as well as nonlinear compressive deformation. The influence of LSB connector slip load $F_{sb}$ on response of the walls is studied, with particular focus on the proposed design slip load of 80 kN (18 kips), chosen based on results indicating optimization of the structural response in the Type B1 wall at this magnitude of $F_{sb}$.

The maximum slip load of 640 kN (144 kips) represents a practical upper limit to the strength of the connector anchorage, when failure of the anchors cannot realistically be prevented prior to bolt slip. Moreover, the results reveal that, at $F_{sb} = 640$ kN (144 kips), the wall is essentially an elastically coupled shear wall, since no slip in the vertical joints is observed. An isolated or uncoupled shear wall is represented by $F_{sb} = 0$.

Two forms of Type B horizontal joints are considered. Reinforced
Type B1 joints allow dowel action of the vertical steel which develops shear resistance once slip is initiated, thereby reducing the level of shear slip induced in the horizontal joints. In the unreinforced Type B2 joints on the other hand, a simple gap-slip element possessing zero tensile stiffness across the joint models behavior; thus, shear resistance is based only on the normal force and the coefficient of friction.

Comparison of Behavior for Types A and B1 Horizontal Joints

Fig. 4 illustrates the differences in selected structural responses for the two joint types with slip load varied over the range $F_{sh} = 0$ to 640 kN (0 to 144 kips). Maximum values of compressive panel stress, total base shear and top lateral displacement for 7 seconds of analysis are shown.

The data demonstrate improved behavior for the wall possessing Type B1 horizontal joints with respect to both maximum panel stress and base shear, while maximum top displacement of the wall increases for the Type B1 joints. However, with a maximum displacement of 64 mm (2.52 in.) for $F_{sh} = 80$ kN (18 kips), the drift index (ratio of maximum top displacement to overall building height) for the 10-story building is under $\lambda_{400}$; thus, the increase in displacement does not represent an unacceptable feature of behavior.

With the exception of panel stresses in Type B1 walls, significant reductions in response magnitudes are noted over both uncoupled ($F_{sh} = 0$) and fully coupled ($F_{sh} = 640$ kN (144 kips)) walls, employing LSB connectors with appropriate slip load. Walls with Type A horizontal joints show definite optimum response at $F_{sh} = 160$ kN (36 kips). [Although $F_{sh} = 160$ kN (36 kips) is taken herein as providing optimum response for the Type A structure, only small changes are noted in the magnitudes of base shear and top displacement with the use of $F_{sh} = 320$ kN (72 kips)].

Type B1 horizontal joint behavior, on the other hand, does not allow an as well defined optimum slip load. In contrast to Type A walls, maximum panel stress is not affected by $F_{sh}$, showing less than 5 percent difference between any two values of $F_{sh}$. Base shear is minimized at $F_{sh} = 40$ kN (9 kips), whereas top displacement is least at $F_{sh} = 320$ kN (72 kips). A compromise choice of 80 kN (18 kips) is more satisfactory, reducing base shears by 35 percent from that for elastically coupled walls and decreasing top displacement by 38 percent from that for uncoupled walls. Thus, the LSB jointed Type B1 wall exhibits the best features of the two alternatives, namely, a base shear 15 percent less than the low base shear of uncoupled walls and a top displacement only 13 percent greater than for fully coupled walls.

Based on the above observations related to Fig. 4, it is evident that LSB jointed walls (with or without the action of horizontal joints) exhibit optimized seismic response.

Corresponding envelopes of maximum response over the height of the structure are presented in Fig. 5 for

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* Reference 1 reports the optimum as 640 kN (144 kips) per story, thus comprising two LSB connectors.
the same response parameters as in Fig. 4; included also is the LSB connector slip. Presented is the response for Types A and B1 horizontal joints at optimum LSB slip loads, i.e., $F_{sb} = 160$ and $80$ kN (36 and 18 kips), respectively.

As predicted by the peak response values of Fig. 4, the Type B1 jointed walls show considerably improved behavior in terms of story shears at all levels [Fig. 5(a)], although this is accompanied by the expected increase in lateral displacements of Fig. 5(b). Maximum panel stresses are quite similar in magnitude except at the base, as seen in Fig. 5(c).

As expected for cantilever structures, these stresses increase toward the base but are noted to remain well below the panel compressive strength $f'_c = 34$ MPa (4.9 ksi), thus confirming the validity of assuming elastic behavior of the panels. For Type B1 behavior, the horizontal joint compressive strength $f'_c = 14.5$ MPa (2.1 ksi) limits the stress in the panels, and the lowermost level shows that this strength of the Type B1 joints is indeed reached.

Considerably higher magnitudes of LSB connector shear slip over the wall height are shown in Fig. 5(d) for Type B1 walls, indicating a maximum slip of 20 mm (0.79 in.) in the uppermost connector. However, since the friction type LSB connectors are designed to slip without yielding of materials, greater slip permits higher levels of energy dissipation in the vertical joint without accompanying permanent damage. In both wall types, this process is seen to act over most of the wall height. Detailing of the vertical joint to accommodate the differential movement of the walls, and the provision of a slot length sufficient to permit unrestrained bolt slip, ensure efficient LSB connector action.

**Effect of Vertical Reinforcement in Type B Joints**

The Type B1 horizontal joints examined thus far contain vertical reinforcement sufficient to develop dowel action and thus restrict shear slip in the joints. The influence of such reinforcement is now examined by comparing the behavior of the Type B1 prototype structure with the Type B2 version wherein no vertical steel is provided.

Fig. 6 compares the responses of the two Type B horizontal joints for $F_{sb} = 80$ kN (18 kips). Story shears, lateral displacements and maximum panel stress exhibit substantial reductions over the height of the wall when unreinforced Type B2 horizontal joints are adopted. The most dramatic effect of lack of vertical steel, however, is observed in Fig. 6(d) which shows that slip in the LSB connectors is nearly eliminated at all levels.

The above-implied increase in horizontal joint slip for unreinforced Type B2 joints is evident in Figs. 7(a) and 7(b), where envelopes of maximum slip and accumulated slip are presented. While less than 1 mm (0.039 in.) of maximum slip is noted at all levels for the reinforced wall, maximum slip of 3 to 5 mm (0.12 to 0.20 in.) is incurred at Levels 1 through 7 in the unreinforced wall, peaking at 11 mm (0.43 in.) at the uppermost joint. While this large increase in horizontal slip at the top of the structure may appear surprising, similar behavior has been reported by Schricker and Powell for precast walls consisting of a single panel stack. Corresponding increases are observed for accumulated slip, measuring the amount of energy dissipation.

![Fig. 6. Effect of vertical reinforcement on response of Type B walls. Note: 1 MN = 225 kips; 1 MPa = 0.145 ksi; 1 mm = 0.0394 in.](image-url)
pated by this mechanism.

Story shears are higher near the base and thus energy dissipation as a result of horizontal joint slippage assumes greater significance at the lower levels. Accumulated slip in the unreinforced joints is 300 to 400 percent greater at Levels 1 to 3 than for vertically reinforced joints. The apparent conflict between the lack of energy dissipation due to virtual elimination of vertical joint slip as noted in Fig. 6(d) and the observation that overall structural responses, such as story shears and panel stresses, are nonetheless improved upon [Figs. 6(a) and 6(c)] is thus resolved by the foregoing compensation in energy absorption in the horizontal joints.

Although effective and therefore attractive, allowing large slip in the horizontal joints in order to increase seismic energy dissipation, as opposed to vertical joint slip, introduces a number of problems.

Panelized wall systems, in particular those employing platform type horizontal joints, are apt to suffer permanent damage as a direct consequence of the large magnitude of slip in these joints. Elastic corrective forces which would otherwise restore the wall to its original position following vertical joint slip are not available during lateral panel slippage; in addition, the in-plane slip of the horizontal joints is liable to cause out-of-plane displacement of the wall panels.

Tests by Mattock have shown that cyclic loading at large amplitude of slip precipitates deterioration of the joint interfaces; thus, the subsequent loss of joint shear strength can seriously compromise both the serviceability and integrity of LP walls. The analyses performed herein showed only six load reversals in shear across the Level 1 reinforced horizontal joint, whereas over 18 cycles were noted in the unreinforced joint. Combined with the larger amplitude of slip, degradation of the horizontal joints is therefore substantially more pronounced in the unreinforced walls.

The envelopes of gap opening in the horizontal joints, shown in Fig. 7(c), indicate much higher values for reinforced Type B1 horizontal joints compared to the unreinforced Type B2 joints. Magnitudes of joint opening are under 0.6 mm (0.02 in.) at all levels for the unreinforced wall, whereas a maximum of 4.2 mm (0.17 in.) at the base is observed for the reinforced wall. Tests have shown that forces in panelized wall systems can be reduced by such a horizontal joint opening through a base isolation effect, even though accompanied by very little energy dissipation. The high level of gap opening for the reinforced wall thus contributes to the reduction in story shears shown in Fig. 5(a) where the continuous, rigidly jointed Type A wall without joint opening experiences higher magnitudes of induced force.

**Bolt Bearing and Tensile Loads in LSB Connectors**

Fig. 6(d) indicates a maximum slip of 20 mm (0.79 in.) in the LSB connectors for reinforced Type B1 walls with \( F_{sb} = 80 \text{ kN} \) (18 kips), and thus a slot length of 65 mm (2.56 in.) is sufficient to accommodate the expected shear slip for M22 (7/8 in.) bolts. The effect of limiting the slip distance in the connectors and consequently permitting bearing of the bolts is shown in Fig. 8, where a maximum slip length of 15 mm (0.59 in.), or 75 percent of the expected unrestricted maximum, is imposed on the LSB connectors.
Although data are not presented herein, most overall response parameters were noted not to be adversely affected by restricted bolt slip; only story shears and horizontal joint slip exhibit noticeable changes. Figs. 8(a) and 8(b) show shear force and horizontal slip to increase by 17 and 64 percent, respectively, at the base.

The most direct effect of the restricted slot length is, of course, to limit the slip in the connectors to 15 mm (0.59 in.), as Fig. 8(c) shows; this maximum slip is reached from Levels 5 through 10, where bearing of the bolts against the slot edges occurs.

The expected result is a dramatic increase in the shear load on the LSB connectors, as the design slip load of 80 kN (18 kips) is exceeded. Fig. 8(d) shows load levels above 160 kN (36 kips) (the yield strength of the connector anchorage) at all levels where bearing occurs, and permanent damage to the vertical joint due to yielding is to be expected.

Supplying more and (or) larger stud anchors, with shear strength greater than 220 kN (49.5 kips), avoids material yielding when bearing occurs. However, the results obtained herein are based on unlimited elastic strength of the anchors. As Fig. 8(d) shows, the shear load which would otherwise be limited to 160 kN (36 kips) if the LSB connector anchorages yield, is approximately 210 kN (47 kips) at Levels 6 and 7, but below the 254 kN (57 kips) shear strength of the two M22 (% in.) bolts provided. The predicted yielding of the anchorage prior to shear failure of the bolts is a desirable feature since the otherwise sequential "unbuttoning" along the vertical joint accompanying bolt failure would invite the increased response associated with vertically uncoupled walls.

The tensile capacity of the LSB connectors is also governed by failure in shear of the two M22 (% in.) bolts, which at 254 kN (57 kips) is less than the concrete pullout capacity of 325 kN (73 kips) for the anchor studs. Fig. 9(a) shows maximum tensile force in the connectors incurred for differing slip load $F_{sb}$. Whereas vertically continuous Type A walls exhibit zero tensile forces, it is evident that the connector tensile capacity becomes an important design consideration for precast walls with Type B1 horizontal joints.

For the optimum slip load $F_{sb} = 80$ kN (18 kips), Fig. 9(b) shows that accidental bearing in the LSB connectors results in only a slight increase in the tensile loads on the connectors. Equally important, the tensile loads incurred in the LSB connectors are well below the failure strength at all levels. The peak tensile load, induced in the uppermost connector, is observed to be 65 percent of the failure capacity of 254 kN (57 kips).

Overall Structural Integrity

Structural integrity of precast walls is important to their serviceability and stability following seismic loading. Control of overall structural responses, such as maximum panel stress and lateral shear force, ensure against collapse of the building; however, minimizing damage to the panel joints is equally important when considering future performance of the structure. The use of LSB connectors is an immediate advantage in this regard, as deformation in the vertical joint is controlled and material yielding is avoided.

The time histories of top displace-
ment for Types A and B1 (reinforced) walls, shown in Fig. 10(a), indicate higher amplitude of vibration for Type B1 walls, with maximum response occurring at approximately 2 seconds. (Peak values were generally unchanged for example cases employing 10 to 15 seconds of the earthquake record, thus confirming use of 6 seconds of excitation as appropriate.) Drift in displacement, an indication of potential instability during the seismic loading, is absent. In both cases, the walls are seen to return almost to their original positions, and no permanent set which may harm future performance of the walls is observed. The time histories of base shear, shown in Fig. 10(b), reveal similar trends in behavior.

Fig. 11 shows the time histories of both the LSB connector shear slip and force at the uppermost level for both wall types. Although a maximum slip of 20 mm (0.79 in.) is observed for the Type B1 structure, with at least eight reversals in slip displacement, damage to the vertical joint is controlled by the design of the LSB connectors. By allowing slip at the predetermined load \( F_{sb} \), a ceiling on the maximum shear load on the connectors is imposed, so that this \textit{a priori} knowledge allows anchorages to be designed not to yield before bolt slip occurs.

Fig. 11(b) shows that the respective slip loads of 160 and 80 kN (36 and 18 kips) are not exceeded, although some residual forces remain at the end of the excitation. However, as indicated by Pall, \(^1\) such residual forces do not affect the future performance of the LSB connectors.

Fig. 12* presents graphically exaggerated deformed shapes of the coupled walls for the three cases of horizontal joints considered: namely, Type A \([F_{sb} = 160 \, \text{kN} \, (36 \, \text{kips})]\); reinforced Type B1 \([F_{sb} = 80 \, \text{kN} \, (18 \, \text{kips})]\); and Type B2 unreinforced \([F_{sb} = 80 \, \text{kN} \, (18 \, \text{kips})]\). For comparison purposes, times corresponding to the representations are not identical for the three cases but are, instead, selected based on the times when maximum lateral displacements of the three structures are reached.

Fig. 12(a) illustrates the continuous nature of the Type A walls, with the predominant deformation limited to the vertical joint. Integrity of the wall is excellent here. For reinforced Type B1 walls on the other hand, Fig. 12(b) shows the dominant role played by the rocking response mechanism which led to the large reduction in story shears noted earlier. Also evident is the increased shear slip in the LSB connectors, indicating the enhanced level of energy dissipation along the vertical joint for this wall.

Fig. 12(c) depicts the deformed configuration of the unreinforced Type B2 wall, which confirms that the lateral displacements are caused primarily by shear slip in the horizontal joints. With negligible vertical joint slip, energy dissipation is thus confined to the horizontal joints. However, as noted previously, large accumulated slip accompanied by an increased number of load reversals compared to the reinforced Type B1 walls invites deterioration of these joints and thus compromises the structural integrity of this system.

Fig. 13 shows the deformed shapes

* The scales for the deformed configurations of the six cases shown in Figs. 12 and 13 are not identical; instead, scales were selected individually in order to highlight differences in the modes of response.
of the coupled walls for the three cases at 6.75 seconds, i.e., 0.75 seconds after termination of the earthquake excitation. As evident in Fig. 13(a), elastic corrective forces in the wall panels are sufficient to return the Type A jointed wall to what is essentially its original undeformed shape; only slight permanent deformation along the vertical joint is observed, approximately 2 mm (0.08 in.) in the topmost LSB connector. The residual lateral displacement of the wall is negligible.

The reinforced Type B1 jointed wall allows for a similar final configuration, with the LSB connectors returning almost to their original positions also [Fig. 13(b)]. As also the case for the Type A wall, this is in contrast to various times during the earthquake when slip in the mechanical connectors is found to be considerable [see Fig. 11(a) for time histories]. Thus, the effectiveness of the LSB connectors to dissipate energy through inelastic joint action, while not incurring damage and at the same time allowing the joint to return almost to its original position, becomes apparent.

However, unlike the strong Type A jointed wall, Fig. 13(b) also shows that the weaker inelastic joints of the Type B1 wall exhibit some residual

Fig. 10. Top displacement and base shear time histories for Types A and B1 walls.
Note: 1 mm = 0.0394 in.; 1 MN = 225 kips.

Fig. 12. Deformed wall configurations at times of maximum displacement.
compressive deformation at the lowermost level. This deformation is the expected result of rocking and is therefore a localized effect; it is confined to the two outer edges of the wall at the base and thus does not imply distress in the joint as a whole.

Furthermore, for \( F_{sb} = 80 \text{ kN} \) (18 kips) the maximum compressive deformation is only 48 and 36 percent, respectively, compared to that for uncoupled \( (F_{sb} = 0) \) and elastically coupled \( [F_{sb} = 640 \text{ kN} (144 \text{ kips})] \) walls. Thus, while some local damage is to be expected in the lowermost horizontal joint, the reduced level is not expected to affect seriously the otherwise enhanced structural integrity of the "tuned" Type B1 wall.

Fig. 13(c), representing the residual deformed shape of the unreinforced Type B2 jointed wall, shows no permanent vertical joint deformation which is as expected, since virtually no slip occurs there. However, relatively large permanent horizontal joint slip is noted at nearly all floor levels, with the most pronounced residual deformation induced at the top of the structure.

**DESIGN IMPLICATIONS**

The earthquake performance of three different types of 10-story pro-

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Fig. 11. Top LSB connector time histories for Types A and B1 walls: (a) deformation; (b) shear force. Note: 1 mm = 0.0394 in.; 1 kN = 0.225 kip.

Fig. 13. Deformed wall configurations at 0.75 seconds after termination of excitation.
totype precast panel structures equipped with LSB mechanical connectors along the vertical joint to improve seismic response has been studied, namely: (1) Type A — no horizontal joint action; (2) Reinforced Type B1 — includes action of vertically reinforced platform horizontal joints; and (3) Unreinforced Type B2 — no vertical reinforcing in the platform joints. Based on the foregoing discussion of the results, the following observations related to the seismic design of such friction-jointed precast structures are noted:

1. For precast walls which are adequately reinforced and possess strong open (Type A) horizontal joints, limited slip bolted (LSB) connections optimize response effectively at a relatively well-defined optimum slip load \( F_{sb} = 160 \text{ kN} (36 \text{ kips}) \) for the prototype structure.

2. For walls with weaker but vertically reinforced platform (Type B1) horizontal joints, on the other hand, the optimum slip load is less well defined and depends on the particular response parameter. Nevertheless, the structure can still be tuned to provide overall optimum response at a connector slip load less than for Type A joints \( F_{sb} = 80 \text{ kN} (18 \text{ kips}) \) for the prototype structure.

3. Over the full range of LSB connector slip load examined, considerably lower panel stresses and story shears are noted for the Type B1 jointed walls, although these are accompanied by larger (but easily acceptable) displacements.

4. Unreinforced Type B2 horizontal joints exhibit large energy absorption capacity through shear slip, thus eliminating both the effectiveness and the need for LSB connectors along the vertical joint. Large reductions in story shears, panel stresses and gap opening arise from this source of energy dissipation. As expected, however, these are accompanied by large increases in maximum and accumulated slip, as well as in the number of slip reversals.

5. In terms of overall structural integrity, both during and following an earthquake, Type A jointed walls are characterized by excellent performance, returning to the undeformed configuration with virtually no permanent deformation. Reinforced Type B1 jointed walls show pronounced rocking at the time of peak response and some permanent, though not large, deformation along the vertical joint. By comparison, unreinforced Type B2 jointed walls respond primarily in non-corrective horizontal shear slip, with insignificant participation of the vertical joint connectors.

6. A design slot length of 65 mm (2.56 in.) for the LSB connectors is adequate for both Types A and B1 horizontally jointed structures. Inadequate length resulting in bearing of the bolts against the connector plate has no major effect on overall response in general. The primary effects are large relative increases in horizontal shear slip in the lowermost joints and, most importantly, a dramatic increase in the load on the LSB connectors. The latter is sufficient to indicate yielding of the prototype connector anchorages, but failure of the prototype LSB connector bolts is not anticipated. Based on the data presented, anchorage and bolt shear capacities should be designed for approximately three times the optimum slip load \( F_{sb} \) to prevent these modes of failure if the required slot length is not provided (either intentionally or accidentally).

7. The tensile capacity of the LSB connectors is also an important design consideration for Type B1 horizontally jointed walls. For a tuned structure this capacity will generally be governed by the shear strength of the LSB connector bolts when headed stud anchors are employed. Here also, three times the optimum LSB slip load will avoid this mode of failure.

Whereas the preceding observations have been obtained from an investigation wherein it was attempted to model the precast systems in a realistic manner, it nevertheless needs to be remembered that the numerical results are applicable only to 10-story buildings of similar properties. Actual design recommendations must await additional studies of structures possessing different characteristics. In particular, the specific values observed for the design parameters such as the slip length may need to be adjusted to suit the structure under consideration.

**CONCLUSION**

The results of this study indicate that limited slip bolted (LSB) connectors placed along the vertical joints of precast large panel structures are an effective means of improving seismic response. First demonstrated in Reference 1 for idealized structures, it has been shown herein that such friction joints can have an equally positive effect on more common types of precast structures. While the present study has considered only 10-story prototype structures, the idealized cases of Reference 1 encompass the range of 5 to 20 stories, and thus predict optimization of seismic response for the horizontally jointed structures of the present study over this range also.

Since a more direct procedure is not available, inelastic dynamic analysis such as employed herein can be used to determine the LSB connector slip load which is required to assure optimum response. As noted in Reference 1, and also supported by results generated during the current study but not presented, the optimum connector slip load is independent of the seismic excitation itself in terms of both the frequency content and the level of intensity. Thus, a single earthquake record suffices in this procedure to optimize the design of the proposed friction jointed precast structure.

Finally, whereas the numerical results have shown good performance of the proposed LSB connectors, it is recognized that experimental investigations such as shaking table tests need to be conducted. This is essential not only to verify the analytical modeling but also to confirm the proposed design procedure.

**ACKNOWLEDGMENT**

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APPENDIX A — ANALYTICAL MODELING OF JOINTS AND PANELS

![Diagram](image)

Fig. A1. Force-deformation relations for Type B horizontal joints.

REFERENCES

7. CPCI Metric Design Manual: Precast and Prestressed Concrete, Canadian Prestressed Concrete Institute, Ottawa, Ontario, Canada, 1982.
Nonlinear spring type joint elements were attached to the library of the three-dimensional general purpose computer program ANSR-I.\(^8\) Fig. A1 shows the gap friction constitutive relations adopted for the Type B horizontal joints.

Since the modeling of those joints generally follows the idealization described by Kianoush and Scanlon, a fuller explanation for these relations is available in References 16 and 17. Along the vertical joint, on the other hand, the LSB connectors were modeled by the force-deformation relations shown in Fig. A2. The numerical values employed for the parameters are listed in Tables 1(b) and 1(c).

The manner in which the above-mentioned spring elements modeling the joints were assumed to interconnect with the wall panels is shown in Fig. A3. The adequacy of the four rectangular elements to represent the behavior of individual wall panels was confirmed by comparison with results employing finite element meshes of smaller size.

### APPENDIX B — NUMERICAL DESIGN EXAMPLE

**Determination of Optimum Slip Load**

After examining alternative methods, the procedure suggested in Reference 1 for establishing the optimum slip load was adopted. It consists of optimizing the energy dissipated by the LSB connectors of a pair of coupled walls assuming that each wall acts as a continuous cantilever and that the connectors slip over the full height of the structure under static triangular lateral loading as shown in Fig. B1. The equivalent distributed coupling shear provided by the slipping connectors is given by:

\[
\sigma = 2n F_{sh}/H
\]

where \(n\) is the number of stories and \(H\) is the height of the structure.

For total distributed load \(W\), it can be shown that the maximum stress occurring at the edges of the wall bases is:

\[
\sigma = \frac{2WH}{b^2t} - \frac{2q_s H}{bt}
\]
Similarly, the energy dissipated by the LSB connectors through friction along the vertical joint is given by:

\[ U_f = \frac{11q_u WH^3}{10EBt^4} - \frac{8b q_u H^3}{3Eb^2} \]  

(3)

For maximum energy dissipation, the condition that \( dU_f/dq_u = 0 \) yields:

\[ q_u = \frac{33W}{160b} \]  

(4)

Employing Eqs. (1) and (2) leads to Eq. (4), or alternatively expressed in terms of the optimum connector slip load \( F_{sb} \):

\[ F_{sb} = 0.065 \sigma_o bt/n \]  

(5)*

Since Eq. (5) is based on continuous cantilever action of the walls, it is suitable for the design of Type A walls. For Type B1 walls, the presence of the horizontal joints reduces the optimum connector slip load to approximately one-half. Thus, for Type B1 walls:

\[ F_{sb} = 0.0325 \sigma_o bt/n \]  

(6)

For the prototype Type A walls of this study, the peak stress \( \sigma_o \) is suitably set to 0.6 \( f' \) of the panels. This yields:

\[ F_{sb} = 0.065 \times 17500 \times 7.3 \times 0.2/10 = 165 \text{ kN} \]

for the Type A walls compared to 160 kN established by dynamic analysis.

For the Type B1 prototype walls, on the other hand, \( \sigma_o \) should correspond to the compressive strength of the horizontal joints (i.e., \( f' = 14.5 \text{ MPa} \) in this study). Thus, the optimum connector slip load becomes:

\[ F_{sb} = 0.0325 \times 14500 \times 7.3 \times 0.2/10 = 70 \text{ kN} \]

which compares well with \( F_{sb} = 80 \text{ kN} \) reported in the parametric study.

**Suggested Design Procedure for Type B1 Structure**

Based on the design implications reported in this paper, the following example calculations apply for the reinforced Type B1 structure:

1. Determine the required slip load per LSB connector. Based on previous calculations:
   
   Required \( F_{sb} = 0.0325 \sigma_o bt/n = 70 \text{ kN} \)

2. Select the size of bolts. To prevent shear failure if bolts come into bearing following slip:
   
   Required shear capacity per connector = \( 3 \times F_{sb} = 3 \times 70 = 210 \text{ kN} \)

   From Reference 9, using two M22 A325 bolts:
   
   Shear capacity = 2 x 127 = 254 kN

3. Determining the required slot length. To prevent bearing of the bolts following slip:
   
   Required slot length = bolt diameter + 2x (maximum slip) = 22 + 2(20) = 62 mm
   
   Use slot length = 65 mm.

4. Select the anchorage embedment based on pull-out capacity:
   
   Required pull-out capacity = \( 3 \times F_{sb} = 3 \times 70 = 210 \text{ kN} \)

   From Reference 7:
   
   Pull-out capacity = \( \phi A_o (0.33 \lambda V') \)

   where
   
   \( A_o = \sqrt{2} \pi \rho (l_r + d_h) \)
   
   \( f' = \text{panel strength} = 34 \text{ MPa} \)

   \( \lambda = 1.0 \)

   \( \phi = 0.7 \)

   \( l_r = \text{anchor length} \)

   \( d_h = \text{head diameter} \)

   Using 155 mm long stud anchors with 32 mm diameter heads, the pull-out capacity is 325 kN.

5. Determine the size of anchors to be used based on shear capacity:
   
   Required shear capacity per connector = \( F_{sb} = 70 \text{ kN} \)

   From Reference 7:
   
   Shear capacity per anchor = \( \phi m A_s F_y \)

   With \( \phi = 0.7 \), \( m = 1 \) and \( F_y = 400 \text{ MPa} \) and 19.1 mm diameter stud anchors:

   Shear capacity using two anchors = \( 2 \times 80 = 160 \text{ kN} \)

6. Confirm that anchors yield before pull-out:
   
   Pull-out capacity = 325 kN

   Shear capacity = 160 kN

   Thus, ductile shear failure precedes pull-out.

7. The final design details of the LSB connectors are shown in Fig. B2.

**METRIC (SI) CONVERSION FACTORS**

<table>
<thead>
<tr>
<th>Metric</th>
<th>Conversion Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 m</td>
<td>3.28 ft</td>
</tr>
<tr>
<td>1 kN</td>
<td>0.225 kip</td>
</tr>
<tr>
<td>1 mm</td>
<td>0.0394 in</td>
</tr>
<tr>
<td>1 MPa</td>
<td>0.145 ksi</td>
</tr>
</tbody>
</table>

* This corresponds to a slip load per story of 0.130 \( \sigma_o \), \( bt/n \) instead of 0.176 \( \sigma_o \), \( bt/n \) as reported in Reference 1. The discrepancy is due to a mechanical error in the derivation given in Reference 1.

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