Friction Joints for Seismic Control of Large Panel Structures

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Ground motions arising during earthquakes create oscillating lateral loads on buildings, thereby causing them to sway back and forth with an amplitude proportional to the fed-in energy. If the input energy can be controlled, and its major portion dissipated during building motion, the level of distress can be significantly reduced.

In steel-framed buildings or cast-in-place concrete structures, reliance is placed on the ductility of the structure to dissipate energy while undergoing inelastic deformations. In large panel construction, it is difficult to develop flexural ductility due to the limited continuity in the vertical steel, and although in some European systems the precast panels are jointed to give vertical continuity, the suitability in general of this type of construction for seismic regions is often questioned. The current seismic codes

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are based on the premise of ductility, and impose severe penalties on structures not possessing it, thereby acting against the adoption of panelized structures.

On the other hand, such structures have been constructed in earthquake zones in the Soviet Union, Romania, Cuba, Japan, and their use is gradually spreading.[1-4] Inspection after severe earthquakes has provided enough evidence to show that large panel buildings, which have been designed for earthquake resistance, experience minimum distress.[1,5] While brick and framed buildings failed or were badly damaged, in panelized buildings usually only the joints between the panels developed cracks. Several full-scale and large-scale models of panelized buildings, tested under simulated earthquake loads, have confirmed these findings.[6-9]

Large panel structures are thus seen to be capable of meeting the requirement of safety and damage control, and the question arises as to how these structures, in which the development of flexural ductility is limited, could perform so well. It now appears that the overall energy dissipating capability of the structure is the key factor for its survival rather than just the presence or absence of ductility.

Synopsis

Construction with large precast concrete panels is widely adopted throughout the world. However, in seismic regions, such structures are often viewed with suspicion because of the serious problems posed by the traditional jointing procedures.

A solution to these problems is proposed in the form of friction joints devised to dissipate energy during severe seismic excitations. By locating these connections in the vertical joint lines only, permanent deformations and damage can be minimized.

Nonlinear time-history dynamic analysis has been used to study and to demonstrate how a building can be "tuned" so as to obtain optimum seismic response. The proposed joints act, in effect, both as safety valves and structural dampers.

Energy Dissipation Mechanism

During actual and simulated earthquakes, the damage in large panel structures is usually along the joints with little damage in the panels themselves. Cracking and slipping along these planes of weakness provides a means for energy dissipation, comparable in effect to that due to inelastic yielding in ductile structures. These planes of weakness are also responsible for introducing a nonlinear behavior to the overall building system, while the large panels themselves remain in the elastic range. Thus, the joints are in fact the only location where energy can be dissipated and, hence, these very planes of weakness, properly harnessed, can be used to further improve the seismic response. The challenge, therefore, lies not only in providing joints of sufficient strength but in maximizing their capacity for energy dissipation.

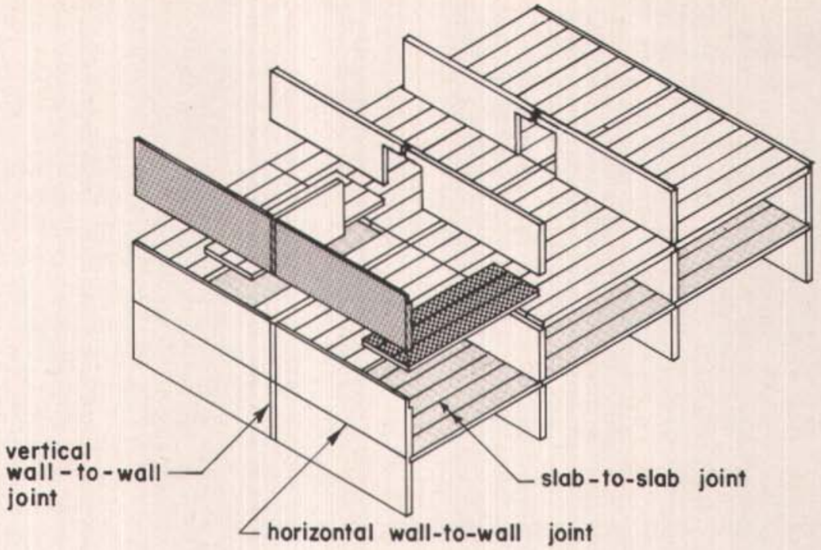


Fig. 1. Types of joints in large panel buildings.

Selection of Joint Location

In large panel construction, there are basically three locations of joints: between floor panels, horizontally between wall panels, and vertically between wall panels (Fig. 1).

As the forces within floor diaphragms are small, there is little probability of slippage or energy dissipation in the floor joints.

The horizontal joint between wall panels is not a desirable location for energy dissipation, because the necessary sliding movements will occur only at the most highly loaded planes, while the deformations will be permanent as there are no corrective elastic forces acting to straighten the building (Fig. 2b). A pure rocking type motion, i.e., opening and closing of joint as shown in Fig. 2c, in a typical horizontal joint, even if post-tensioned, does not cause energy dissipation.[10,11] Furthermore, the concentration of vertical and lateral forces at the corners of panels, associated with rocking motion, may cause failure in either the connection or the panel.[11,12]

There remains the vertical joint between wall panels. This appears to be by far the most suitable location of a mechanism for energy dissipation. Unlike horizontal joints, the vertical joints, after slippage to dissipate energy, return to their original alignment under the elastic action of the cantilevered shear walls, with little or no permanent deformations (Fig. 2d).[13-15]

Also, when the vertical joints slip, the overall building rigidity is reduced, thereby lengthening the effective period of the building, which may be beneficial. Even in extreme loading cases, the failure of a few connections is not likely to threaten the overall stability of the structure as these are not the gravity load carrying joints. Other researchers have also concluded that vertical joint lines are the most logical choice for energy dissipation.[16,17]

The vertical joints that may be utilized are the continuous joints between end walls, the connections between corridor lintels and the right angle joints between wall panels in I, T, L, C, and box sections around elevators or stair shafts.

Joint Design

For the vertical joints to function as an efficient means of energy dissipation, they should possess

1. Elasto-plastic behavior;
2. Stable hysteretic characteristics over the number of cycles expected in a severe earthquake;
3. An ability to accommodate relatively large deformations to dissipate sufficient energy; and
4. A capability to perform these functions without permanent damage.

In addition, the joints must satisfy the normal design functions and carry the usual service loads, such as wind, within the elastic range. None of the jointing systems presently being used meets all the above requirements. Of the various alternatives studied, a mechanical connection appears to be the most suitable.[14, 18]

The Descon-Concordia system[19] uses bolted joints for horizontal wall-to-wall and slab-to-slab joints, but not for the vertical joints, in the panelized buildings first constructed for "Operation Breakthrough" in the United States in 1973. Slotted holes are used to accommodate normal dimensional tolerances. Static tests on the prototype connections were conducted at the U.S. National Bureau of Standards, and a typical result is shown in Fig. 3.[20,21]

The anchorage was more than adequate to develop the joint strength. Dynamic cyclic tests under load reversals were not conducted, as the connections were designed as non-slipping friction type joints. However, the tests showed that with slotted holes the frictional movement could give the desired energy dissipation without causing inelastic yielding of the materials. A slipping bolted connection can, therefore, be engineered to simulate the ideal "elasto-plastic" behavior, with a stable hysteretic character.

The connection chosen consists of

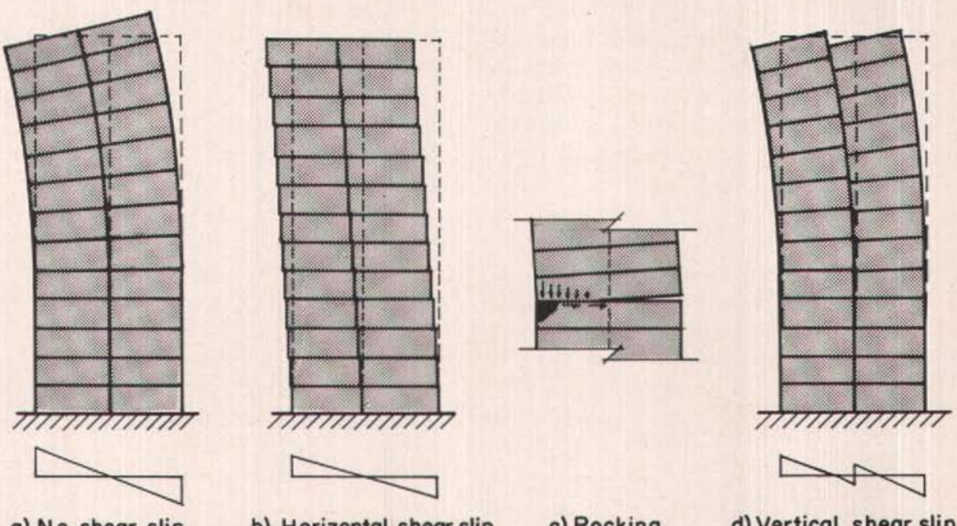


Fig. 2. Modes of deformation of structures for various conditions.

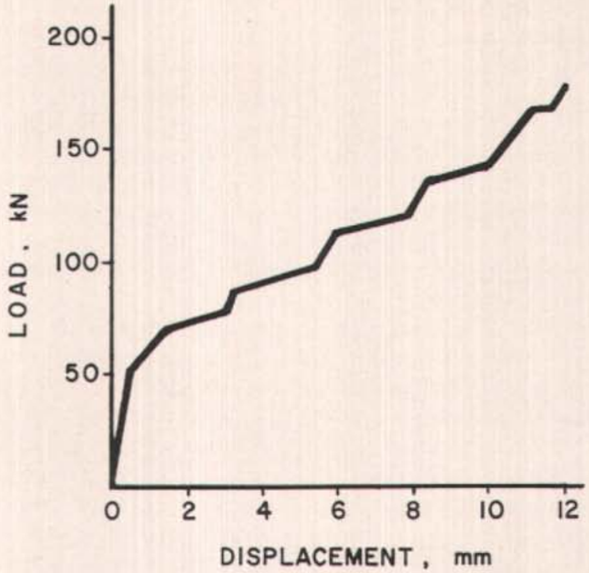


Fig. 3. Load-deformation response of a "Descon-Concordia" bolted joint.

steel plates or sections, with slotted holes, connected by high strength bolts to steel inserts anchored in the concrete panels. The length of the slot accommodates the normal fabrication and erection tolerances with an additional clearance on either side of the bolt to allow the desired slip.

Fig. 4 shows the details of the Limited Slip Bolted (LSB) joints for some of the vertical wall-to-wall connections. The connecting plate is bolted in position during erection but is finally welded on one side to prevent the rotation of the plate when slipping occurs.

The joints are designed not to slip under loads in service, but are expected to slip during severe seismic excitations and so they will not be grouted, but sealed by other appropriate means.

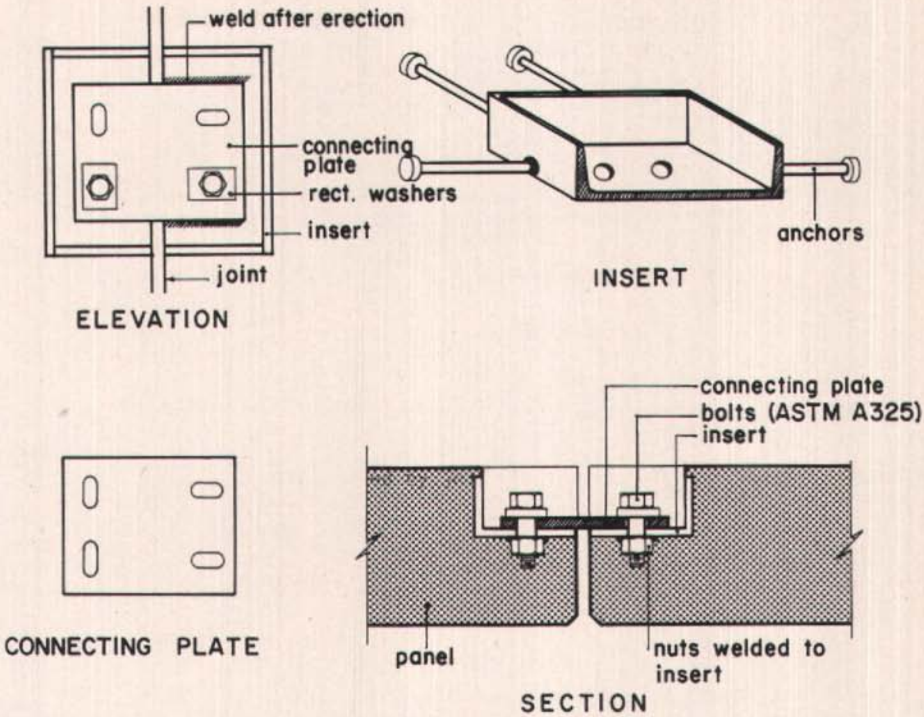
Attention will need to be paid to the details of floor joints and other finishes along the slip planes to accommodate the differential movement of the walls.

Testing of the Joints

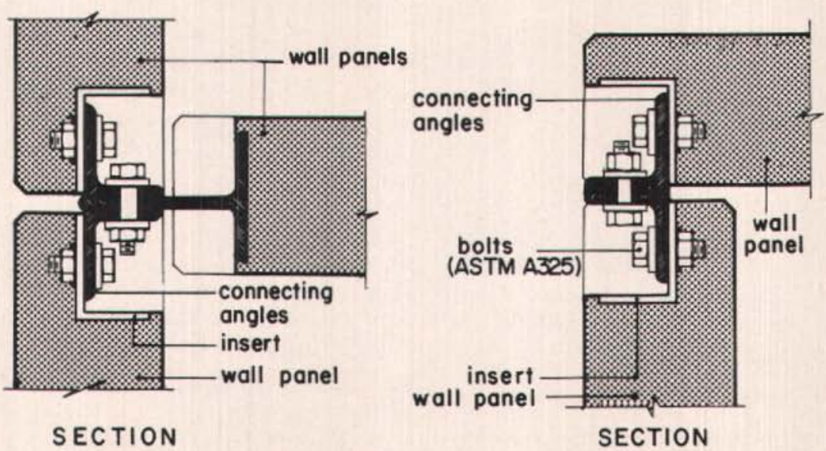
Static and dynamic cyclic tests were conducted on several types of connection, having different faying surface treatments, to evaluate their basic design properties. Load-deformation curves and hysteresis loops, using 12.7 mm (½ in.) diameter high strength bolts (ASTM-A325), are shown in Figs. 5 and 6, respectively.

A predictable and repeatable load is the most important requirement to ensure a predictable response of the structure. Although metalized surfaces showed the highest static slip coefficient and energy dissipation, they are not desirable as the performance is far from predictable. All the externally applied coatings on metal surfaces which were tested were eliminated on this basis.

The best behavior is shown by brake lining pads inserted between steel plates with mill scale surfaces. This joint exhibits a constant, repeat-



a) Simple wall-to-wall joint



b) Corner wall-to-wall joints

Fig. 4. Typical details of Limited Slip Bolted (LSB) joints. Simple wall-to-wall joint (top); corner wall-to-wall joints (bottom).

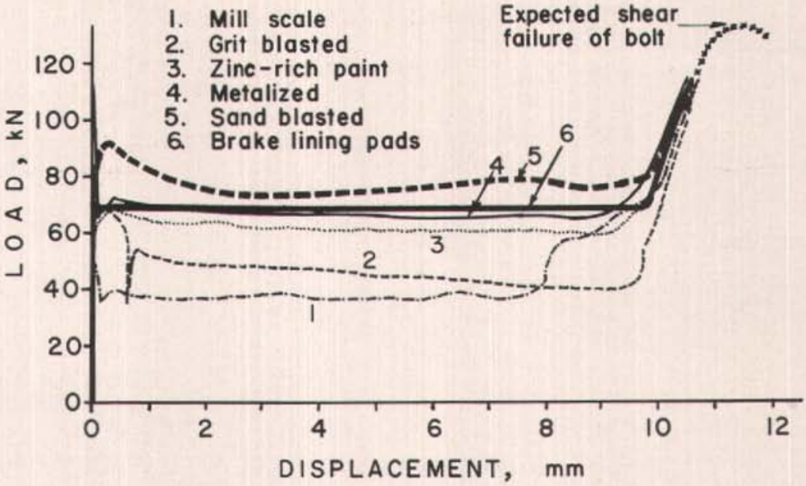


Fig. 5. Load-deformation response of Limited Slip Bolted (LSB) joints.

able slip load and nearly "elasto-plastic" behavior, with negligible degradation. Sand blasted steel surfaces are the second choice.

The load-deformation relationship of the complete joint is elastic up to the point of slipping, after which it is "plastic." Should the bolt reach the end of the slot it becomes elastic again up to the load causing failure in the bolt. The anchorage into the concrete panel is assumed to remain elastic.

Earlier tests[22] indicate that relaxation in bolt tension over a period of 20 years is not more than 8 percent. Using precompressed heavy duty break lining pads, the additional loss in bolt tension has been shown to remain unchanged over a period of one year. Tests to determine the slip load after longer periods are being conducted.

Seismic Analysis

A panelized building assembled using LSB joints will behave in a nonlinear manner when subjected to seismic action severe enough to cause the joints to slip. To investigate the influence of these joints on the seismic response, the typical apartment building plan shown in Fig. 7 was chosen.

The dimensions are representative of a popular crosswall system in large panel construction. The two halves of each end wall are coupled together by using two LSB joints per story height while interior crosswalls are assumed to be coupled at the corridor beam with a single LSB joint. The studies were made for the exterior end walls, which are the most heavily loaded.

Nonlinear time-history dynamic analysis was carried out using the computer program "Drain-2D,"[23] which was modified to incorporate the behavior of LSB joints. The earthquake record of El Centro 1940, north-south component, was used as it is reasonably symmetric. It is known that different earthquake records, even though of the same intensity, may give widely varying structural responses, and values obtained using a single record may not be conclusive.

The analysis was conducted for a duration of 7 seconds, which includes

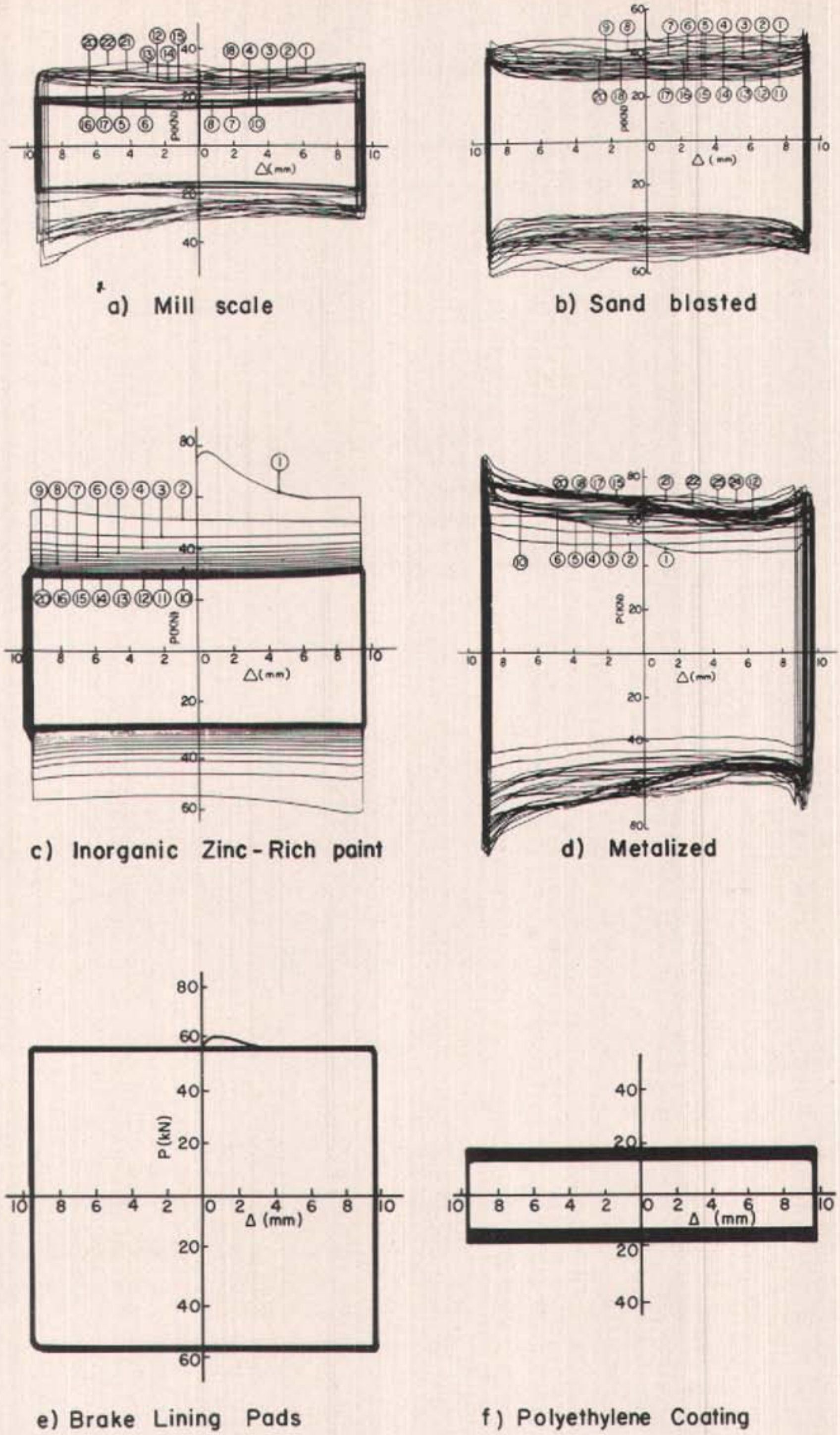


Fig. 6. Hysteresis loops of Limited Slip Bolted (LSB) joints.

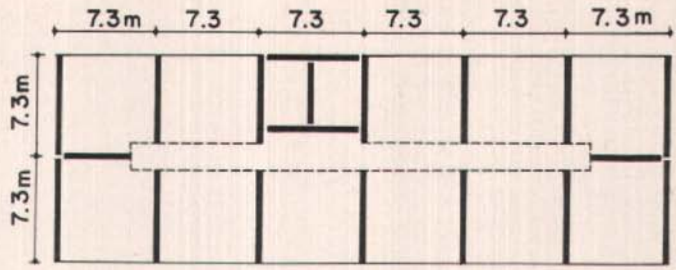


Fig. 7. Plan of a typical apartment building.

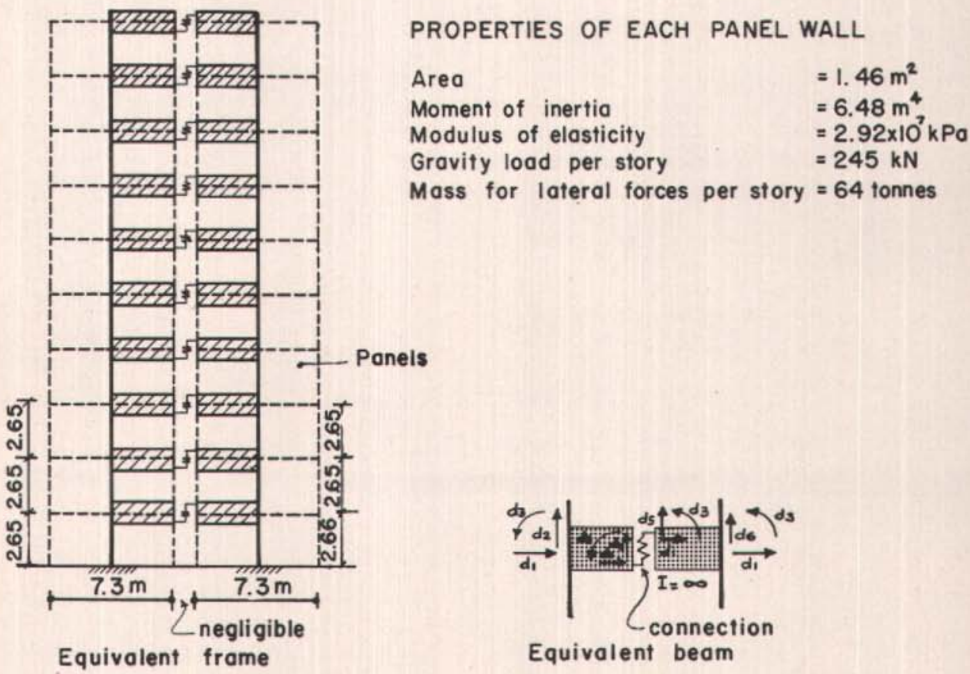


Fig. 8. Structure idealization of LSB jointed wall.

the most severe motion, followed by zero acceleration for 2 seconds to allow the structure to come to rest. An integration time step of 0.01 seconds was used in all the analyses.

Modeling Assumptions

The complete dynamic behavior of a panelized building is a complex problem. To reduce its size, and to isolate the vertical LSB joints in order to examine their behavior and influence, the following assumptions are made:

1. Vertical panel walls are considered as continuous elastic cantilevers. Although each cantilever wall includes horizontal joints, it is assumed that gravity loads or post-tensioning (if necessary) produce sufficient friction

to prevent any shear slip or rocking, and nonlinear behavior of the structure is limited to the LSB joints.

2. The floor diaphragms are sufficiently rigid in their own planes to distribute the lateral forces between the walls in proportion to their stiffnesses. This is generally true except, perhaps, for certain proportions of low rise buildings.[24]

3. Mass and stiffness dependent type viscous damping corresponding to 5 percent of critical is assumed for elastic panel walls. Energy dissipation due to hysteretic behavior of LSB joint itself is accounted for in the computer program.

4. The foundations are rigid and soil-structure interaction is ignored.

The above assumptions, although not completely true, are reasonable enough to concentrate the study on the role of LSB joints.

The stresses and forces computed in this study are only for comparison purposes and are not intended to be design values.

Structural Idealization

The coupled walls are idealized as an equivalent wide column frame shown in Fig. 8. Effects of flexural, axial, and shear deformations are taken into account. The two LSB joints in each story height are lumped at each floor level and modeled as ficticious axial elements yielding in tension and compression, using a modified subroutine of a truss element. The full load-deformation relationship for the joint, shown in Fig. 9, i.e., elastic-plastic-elastic-failure, is represented in the subroutine.

Optimization of Seismic Response

For any earthquake motion, the response of a panelized structure is determined by the amount of energy fed in and the energy dissipated. The optimization of seismic response, therefore, consists of minimizing the difference between the input energy and the energy dissipated.

The energy input is basically dependent upon the mass and the natural frequency of the structure. The latter is influenced by the slip load and the stiffness of the joints. With strong joints between two walls, creating a monolith, the natural period of vibration will be t_1, as shown in Fig. 10. Isolated walls will have a longer period, t_2. The introduction of LSB joints will result in a period, t_3, intermediate between t_1 and t_2 which will vary with the slip load and with the amplitude of the oscillation.

The energy dissipated in a vertical joint is proportional to the product of

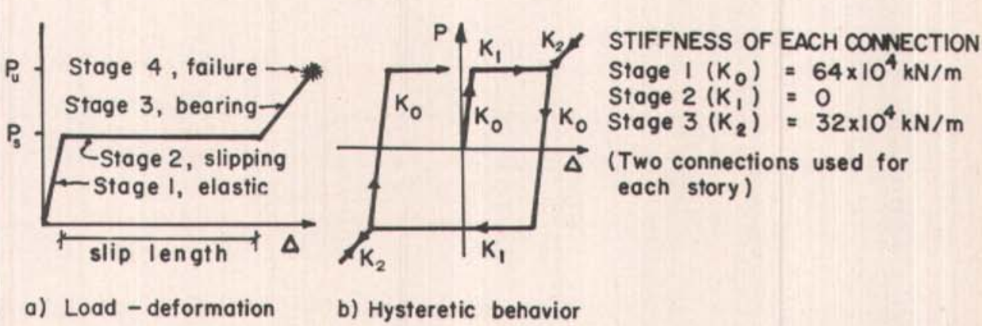


Fig. 9. Idealized behavior of LSB joint.

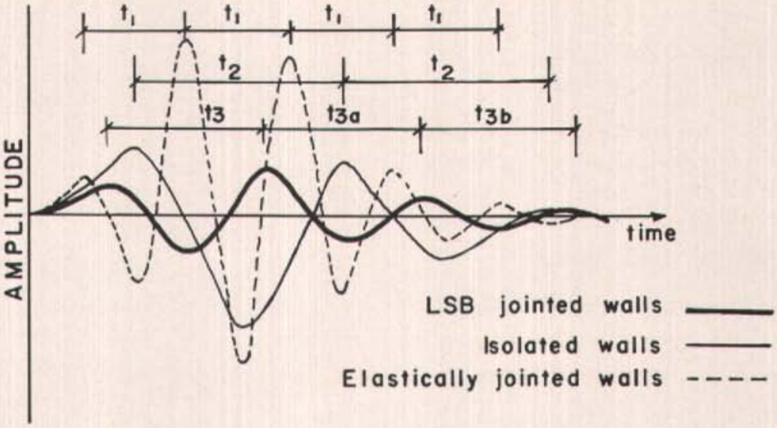


Fig. 10. Oscillation of various types of walls.

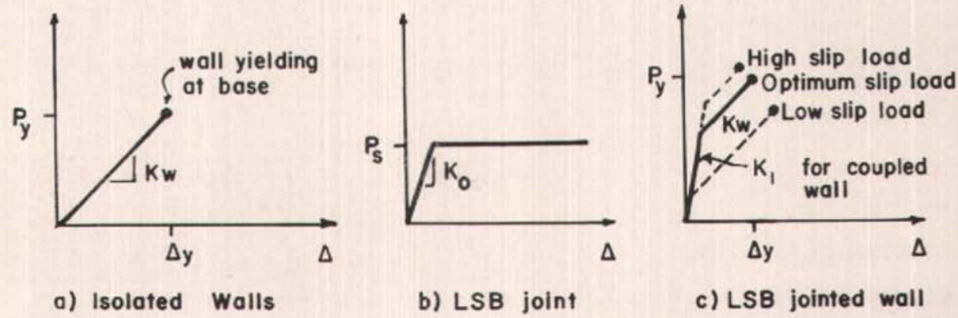


Fig. 11. Effect of joint slip load on load-deformation response of walls.

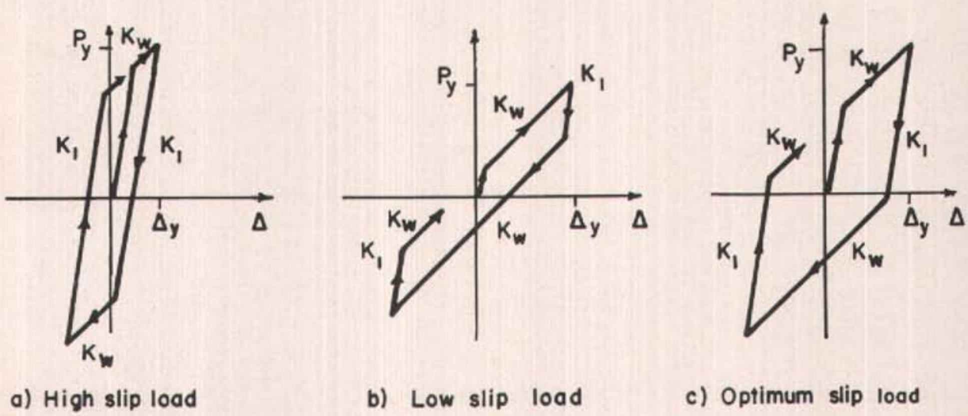


Fig. 12. Effect of joint slip load on hysteretic behavior of coupled walls.

the slip load and the slip travel during each excursion. The effect of the slip load on the deformation of a single story wall is conceptually shown in Figs. 11 and 12. For a very high slip load, the energy dissipation will be zero, as there will be no slip in the joint. If the slip load is very low, the amount of energy dissipated will again be negligible. Evidently, the slip load has to be of an intermediate value to maximize energy dissipation. This is clearly seen in Fig. 12c.

For a single excursion of a symmetric double shear wall, with a specified limiting stress, the maximum energy that can be dissipated by friction in the joint in a quarter cycle is equal to the elastic energy in the walls at maximum amplitude. This is equivalent to critical damping. However, in an actual earthquake, the amplitude varies, with only a few excursions causing the limited stress. The average damping will thus be much less than critical (small amplitudes are elastic). The optimum slip load will vary with the earthquake intensity, and, to some extent, with the type of earthquake spectrum. The value of this optimum is found by direct dynamic analysis.

Softening of the structure, due to slipping of the joints, can mean an invitation to either higher or lower seismic forces, depending upon whether the natural frequency of the building is moved towards, or away from, the dominant frequency of the ground motion. The beneficial effects of energy dissipation must therefore be combined with the positive or negative effects of the prolonged period of vibration.

Parametric Studies

The following parametric studies were carried out to determine the effect of each on the seismic response. The values given for the joints are those for the sum of the joints in each floor:

1. Joint stiffness: (128 to 256×10^4 kN/m);
2. Slot length: (30 to 40 mm for 20-mm bolt);
3. Slip load of the joint per story height: (0, 160, 320, 640, 2560 kN);
4. Building height: (5, 10, 15, 20 stories);
5. Seismic intensity: (0.15, 0.25, 0.33, 0.5 of gravity).

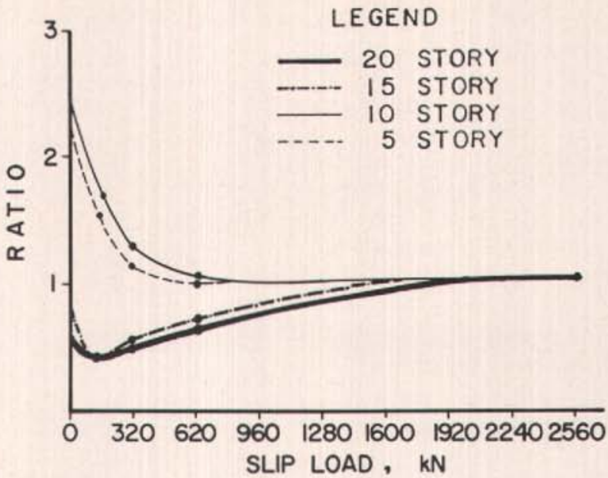
It was observed that:

(a) The variation in the initial stiffness of the joint assembly, within the practical range of such joints, does not cause appreciable change in the response.

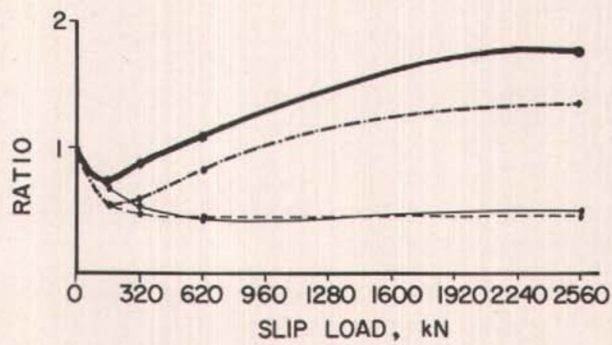
(b) The restriction of the slot length in the connecting plate does not improve the response of the structure, but, on the contrary, could result in permanent damage to the joint and the panels due to the sudden increase in forces caused by the closing of the joint and by the shear failure of the bolt. A sufficient length must be provided to accommodate erection tolerances plus the total movement up to the limiting stress in the walls themselves.

(c) For a given earthquake intensity and building geometry, the slip load of the joint is the variable which most influences the seismic response. By varying the slip load, it is possible to "tune" the response of the structure to an optimum value. The influence of the slip load on the maximum normal stresses at the base and on the maximum deflection at the top is shown in Figs. 13 and 14. Slip loads of 0 to 2560 kN represented unjointed walls and non-slipping elastic joints, respectively.

In general, the slip load which gives the minimum normal stress at the base also gives the minimum deflection, story shear and overturn-



a) Ratio of stress of jointed wall to elastically jointed wall.

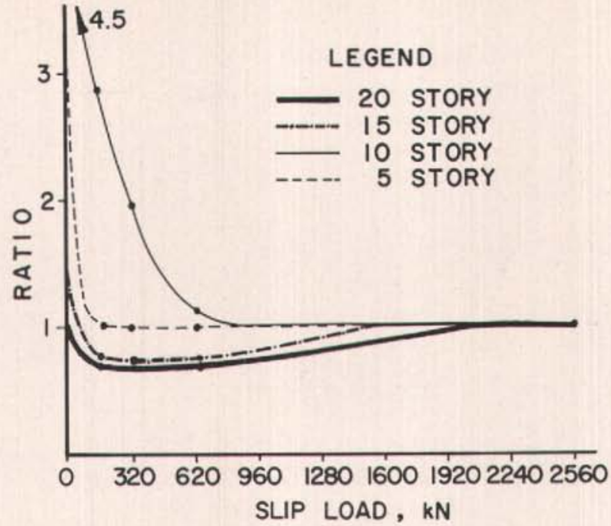


b) Ratio of stress in jointed wall to isolated walls

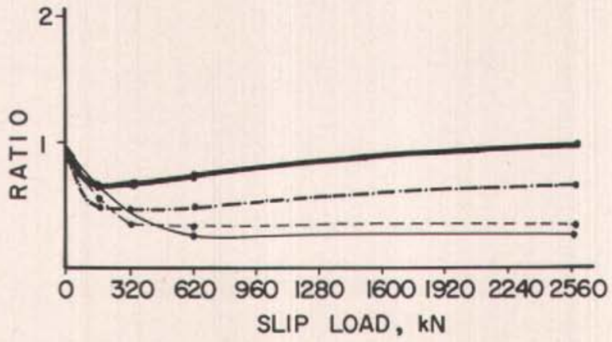
Fig. 13. Influence of joint slip load on normal stresses in walls.

ing moment. The optimum slip load value varies directly with the seismic intensity.

(d) The response of 5- and 10-story walls (where the period is less than 0.5 seconds), is distinctly different from that of 15- and 20-story walls (where the period is greater than 0.5 seconds). In 5- and 10-story walls, the beneficial effect of energy dissipation is countered by the negative effect of the increased seismic force caused by moving the natural frequency towards the dominant frequency of the earthquake, while for 15- and 20-story walls, the benefit of energy dissipation is added to that of reduced seismic force due to the softening of the structure. The limiting building period depends upon the dominant



a) Ratio of deflection of jointed wall to elastically jointed wall.



b) Ratio of top deflection of jointed wall to isolated walls

Fig. 14. Influence of joint slip load on wall deflection.

frequency content of the earthquake motion.

An artificial record, generated at the Massachusetts Institute of Technology[25] to match the Newmark-Blume-Kapur response spectrum, was also used. It was observed that although the response differed widely from that of the El Centro record, the value of the optimum slip load for a given seismic intensity is almost independent of the time history of the earthquake motion.

Discussion of Results

The effectiveness of LSB joints in improving the seismic response of panelized buildings is seen in the

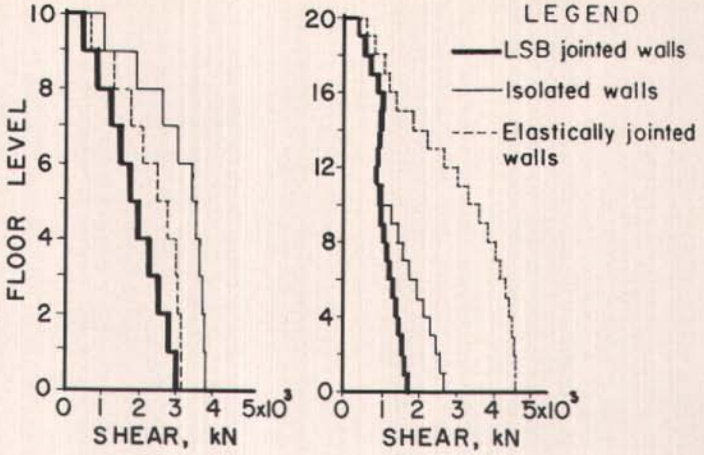


Fig. 15. Shear envelope (0.33g).

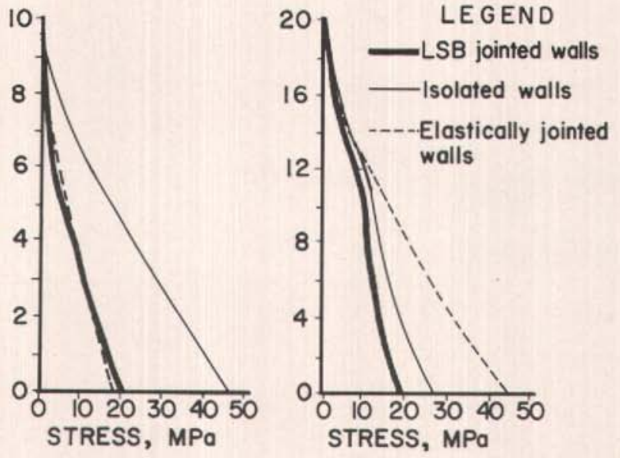


Fig. 16. Normal stresses (0.33g).

comparisons between the results obtained for isolated walls (zero slip load), for walls with strong elastic joints (slip load = 2560 kN), and for walls with the optimum slip load.

Story Shears

In the case of 5- and 10-story walls, the reduction in base shear is about 30 and 25 percent when compared to isolated walls, while little benefit is derived when compared to walls with strong elastic joints.

For both 15- and 20-story walls, the reductions in the story shears are 35 and 65 percent when compared to isolated and elastically jointed walls, respectively. The lower story shear reduces both the stresses in the horizontal joints and the building acceler-

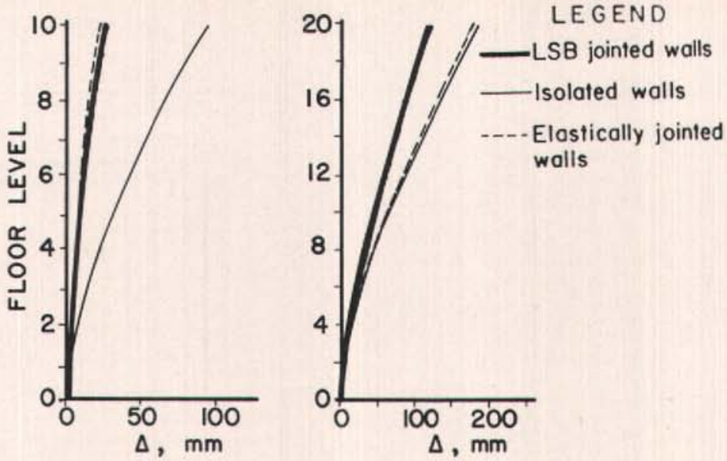


Fig. 17. Deflection envelope (0.33g).

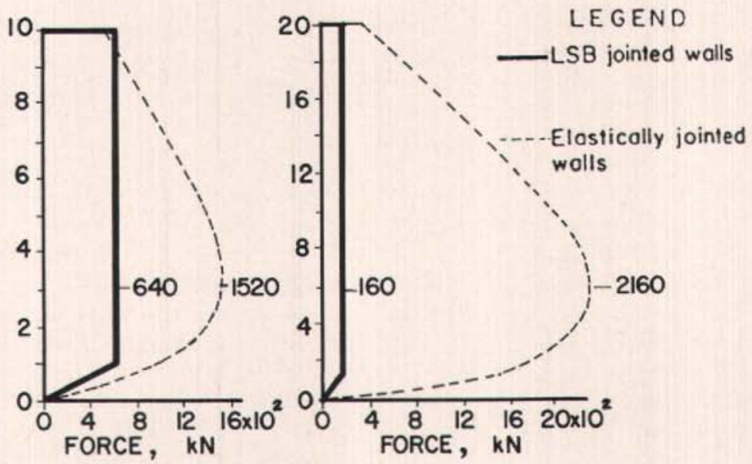


Fig. 18. Forces in joints (0.33g).

ations. Typical story shears for 10- and 20-story walls for a seismic intensity of 0.33g are shown in Fig. 15.

Normal Stresses in Panels

For 5- and 10-story walls, the stress with LSB joints is almost the same as with strong elastic joints, but it is approximately 35 percent of that in isolated walls.

In both 15- and 20-story walls, the stresses with LSB joints are about 40 and 65 percent of those for strong elastic joints and isolated walls, respectively. Normal stresses for 10- and 20-story walls for a seismic intensity of 0.33g are shown in Fig. 16. The effectiveness of LSB joints in reducing the normal stress is shown in Fig. 19.

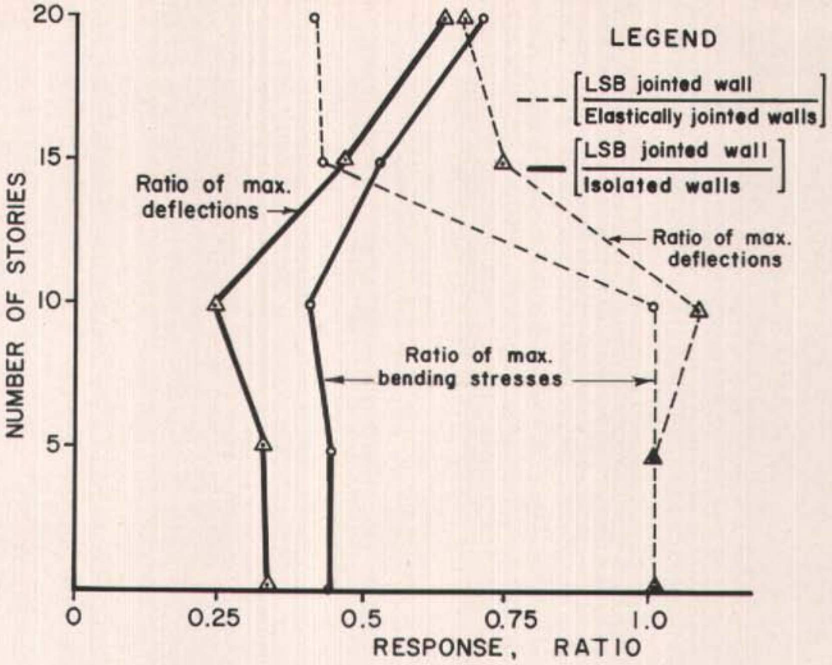


Fig. 19. Relative response of LSB jointed wall for different wall heights (0.33*g*).

Deflections

The deflections of LSB jointed walls of 5- and 10-story height are similar to those for elastically jointed walls, but are about 30 percent of those for the isolated walls.

For 15- and 20-story walls, the deflections are approximately 50 and 70 percent of those for isolated walls and strong elastic joints, respectively. Typical deflection envelopes for 10- and 20-story walls for a seismic intensity of 0.33g are shown in Fig. 17. The effectiveness of LSB jointed wall in reducing the building deflections is shown in Fig. 19.

Forces on the Connections

The distribution of forces in the elastic joints and the LSB joints is shown in Fig. 18. The force in the elastic joints varies over the building height and increases with an increasing severity of earthquake. In the case of LSB joints, as the connections slip, redistribution of forces takes place until they become almost uniform throughout the height.

One of the advantages of the slipping joints is, therefore, to provide a limit to the load, which is a predetermined value independent of seismic intensity. It also allows the full capacity of all the connections to be utilized. Since the force level in LSB joints was far less than that in non-slipping joints, no damage would have been caused to the joint anchorages or the panels.

Time Histories

Typical time histories for the topmost story of 10- and 20-story walls for a seismic intensity of 0.33g are shown in Fig. 20. For the 10-story wall, the peak amplitude of the LSB jointed wall is far less than that for isolated wall but is almost the same as that of

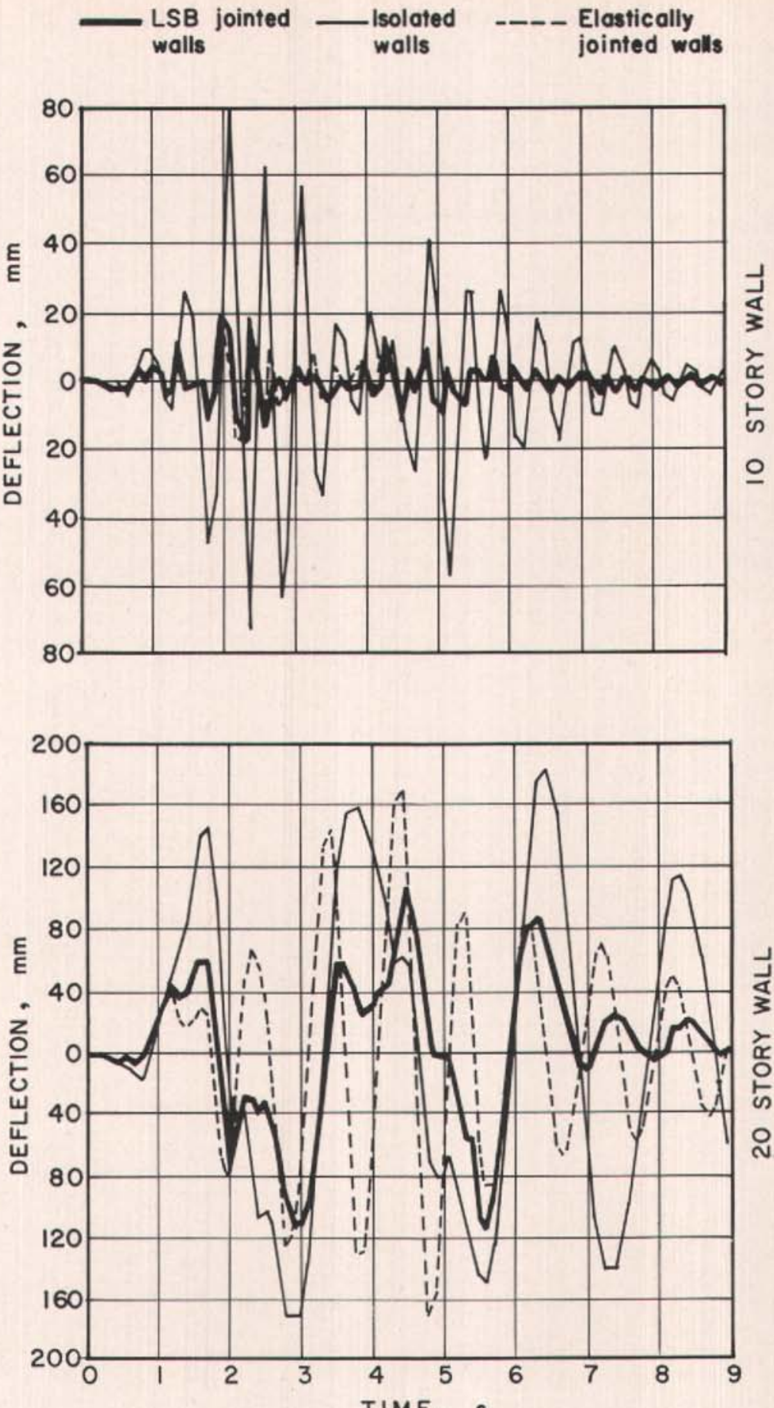


Fig. 20. Time histories for wall deflections at top (0.33g).

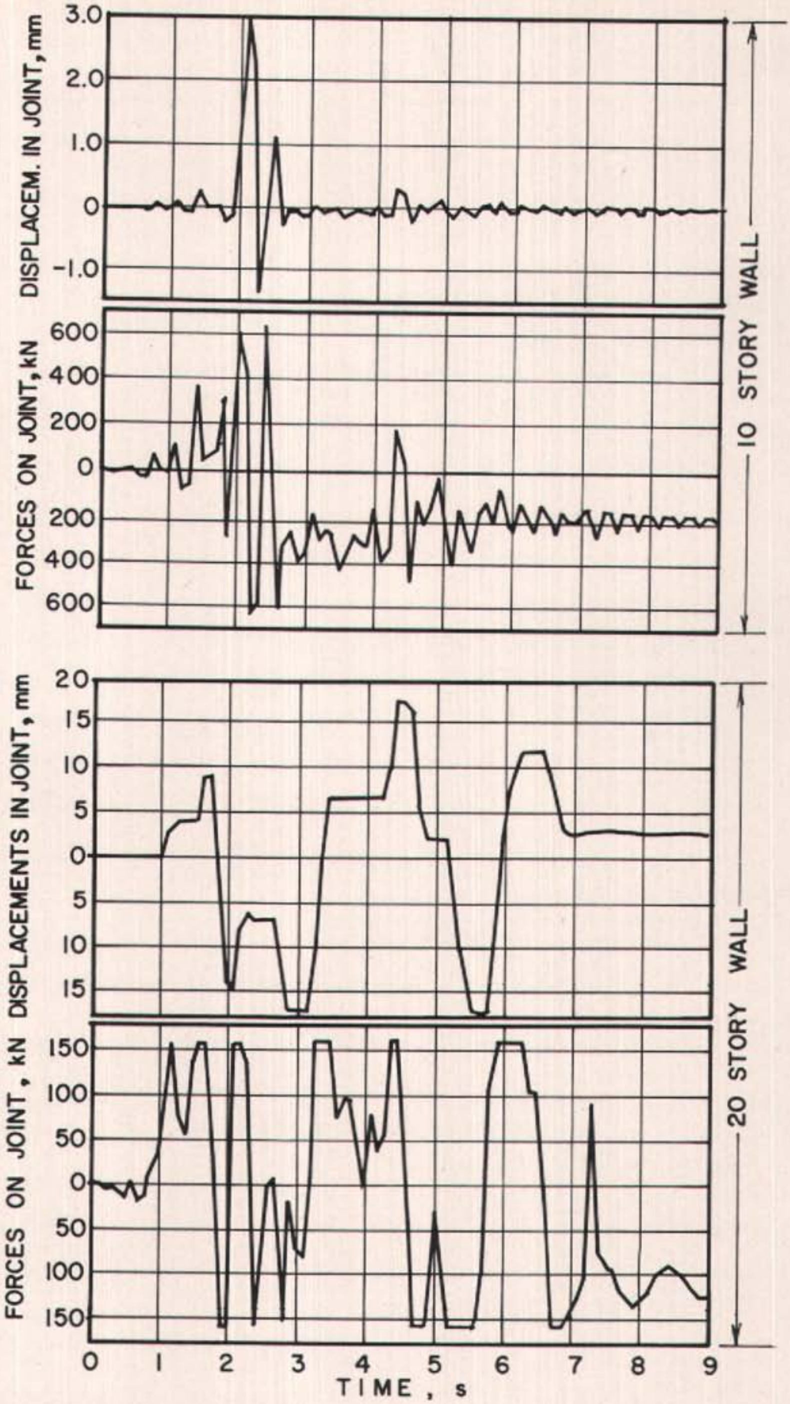


Fig. 21. Time histories of forces and displacement in joints of top story with optimum slip loads, for El Centro earthquake (0.33g).

the elastically jointed wall. However, since the effective period of vibration of the LSB jointed wall is longer than the elastically jointed wall, the accelerations experienced by this wall are less.

The effectiveness of the hysteretic damping of the LSB jointed wall is more clearly seen in the case of the 20-story wall. The amplitude of vibrations and accelerations are considerably less than for both elastically jointed and isolated walls.

It was observed that the effective period of vibration of LSB jointed walls changes with the amplitude of vibration, i.e., with the increasing severity of the earthquake, resonance of the structure is thus more difficult to establish.

Time histories of forces and displacements in LSB joints of the top story are shown in Fig. 21. It is seen that, under the elastic action of the cantilever walls, all the connections return almost to their original position and are ready to face future earthquakes with nearly the same efficiency. There may be some residual forces in the connections at the end of the earthquake, but that does not affect their future performance.

Equivalent Ductility and Hysteretic Damping

Nonlinear behavior has been quantified by the value of an equivalent ductility defined in various ways, but usually viewed as the ratio of the limiting deflection to that at the onset of inelasticity. For the friction joints of the type described, the ratio of the deflection at some limiting stress in the concrete to that at the first slip of a joint, for the optimum energy dissipation at peak amplitude, is approximately 6. During an earthquake, the amplitude varies, and to obtain the optimum energy dissipation for the set of cycles experienced, a lower value of the slip load is required, with a resulting higher value of the above ratio.

No direct means is available to apply the value to obtain an equivalent viscous damping, other than by the complete analysis of the structure damped viscously and damped by friction. The comparison obtained is valid only for the particular case studied at the particular earthquake intensity assumed.

In the present studies, with the optimum slip load for a given intensity obtained from the dynamic analysis, the behavior is equivalent to that resulting from a viscous damping of approximately 20 percent.

Practical Application

The adoption of this device for seismic control will require an arrangement of walls and floors such that independent movement is possible at the slip planes. Pipes, ducts, finishes and seals which might be broken if they cross these planes must be appropriately treated.

To provide some perspective on the relative value of the slip load, a simple analysis is given in the appendix.

Conclusions

The limited slip bolted joint has been developed to meet the requirements of an efficient energy dissipating connection with elasto-plastic behavior and stable hysteretic characteristics.

That it be the vertical lines of connections in which LSB joints are incorporated is of particular importance as:

1. The level of energy dissipation is higher than with horizontal joints since the process acts over the full height.

2. The joint strength can be uniform and all joints can contribute.

3. The building is softened without losing its elasticity and resilience and

recovers with little or no permanent set.

4. The joints act as structural dampers to control the amplitude, and as safety valves to limit the load exerted.

5. The amplitude of vibrations and accelerations are considerably reduced, hence secondary and architectural damage is minimized.

6. The building can be "tuned" for optimum response without resorting to other expensive devices like hydraulic systems or added masses; this "tuning" represents matching the slip load to the anticipated maximum earthquake intensity to give minimum acceleration in the building.

7. There is no yielding of materials involved in the process of energy dissipation, hence no damage is caused.

8. The joints lose little or no tension, and remain without adjustment ready to face the next earthquake with the same efficiency.

The concept of energy dissipation through friction in slipping joints can be easily extended to framed buildings clad with precast concrete curtain walls or infill panels. In this case, either horizontal or vertical joints may be allowed to slip as they carry no gravity loads.

LSB joints may also be used in tall cast-in-place shear walls to increase the flexibility of the otherwise rigid walls and to dissipate energy, resulting in overall improved seismic response. Since large amounts of seismic energy can be dissipated in friction, ductility demand, which is associated with structural and secondary damage, can be considerably reduced.

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* * *

APPENDIX A—OPTIMUM SLIP LOAD

Consider the double cantilever wall of Fig. A1 subjected to a triangular load distribution representing seismic load.

Along the joint between the walls is a uniform shear flux of q per unit length.

The slip between the walls, multiplied by the shear flux, q, is the work done against friction in the joints. This is given by:

$$U_f = q \int_o^h \int_x^h \left[\frac{w}{Eb^2 t}\left(3x^2 - \frac{x^3}{h}\right) - \frac{8qx}{Ebt} \right] dx\, dx = \frac{Qh^2}{E}\left(\frac{11}{20}\frac{wh^2}{b^2 t} - \frac{8}{3}\frac{qh}{bt} \right) \tag{A1}$$

If the maximum stress is limited to σ_u, then:

$$\frac{wh^2}{b^2 t} - \frac{2qh}{bt} = \sigma_u \tag{A2}$$

Using this condition, the maximum energy dissipation will occur when:

$$\frac{qh}{bt} = 0.176\,\sigma_u \tag{A3}$$

This leads to the following relationships at the optimum condition:

$$\text{slip load per floor} = 0.176\, bt\, \sigma_u / n \tag{A4}$$

where n is the number of floors.

$$\text{Energy dissipated in each quarter cycle} = 0.048\,\sigma_u^2 V/E \tag{A5}$$

$$\text{Elastic energy at peak amplitude} = 0.052\,\sigma_u^2 V/E \tag{A6}$$

where $\quad V$ = volume of concrete in one wall = bth

$$\text{Value of } w \text{ at first slip} = 0.47\,\sigma_u b^2 t/h \tag{A7}$$

$$\text{Maximum value of } w = 1.35\,\sigma_u b^2 t/h \tag{A8}$$

$$\text{Stress at first slip} = 0.235\,\sigma_u \tag{A9}$$

$$\text{Deflection at first slip} = 0.065\,\sigma_u h^2/Eb \tag{A10}$$

$$\text{Deflection at limit} = 0.39\,\sigma_u h^2/Eb \tag{A11}$$

The limiting stress, σ_u, may be governed by cracking, the yield in the steel, or some other appropriate limit that can be represented by σ_u.

Consider the end wall of the building plan in Fig. 7 for a 15-story building. The following values apply:

$$b = 7.3 \text{ m}, t = 200 \text{ mm}, n = 15, \text{ and let } \sigma_u = 10 \text{ MPa}$$

Required slip load per floor = 0.176 x 7.3 x 0.2 x 10,000/15
= 170 kN

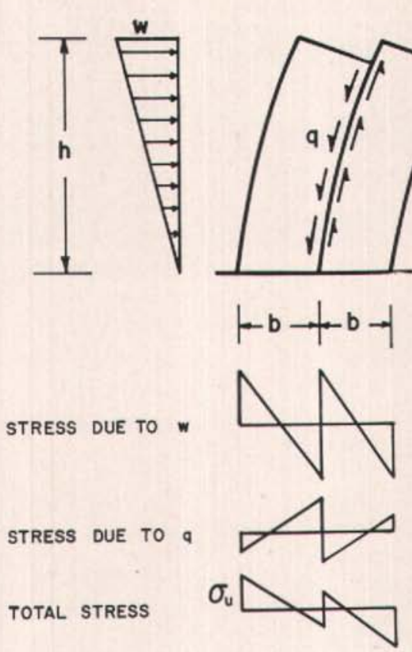

Fig. A1. Stress in double shear wall.

Note that this value is dictated entirely by the proportions of the wall and the limiting stress chosen. This value of the slip load would maximize energy dissipation for those cycles in which the peak stress is σ_u.

This may not minimize accelerations for all earthquake intensities, but it will be the optimum that can be achieved without exceeding σ_u, and will ensure that as much energy as possible has been dissipated by the friction joint before the walls suffer permanent damage. An appropriate value of limiting stress, σ_u, can be obtained by equating the sum of Eqs. (A5) and (A6), i.e., $0.1\,\sigma_u^2\,V/E$, to the peak energy input for a given earthquake intensity.

* * *

NOTE: 1 ft = 0.305 m;
1 in. = 25.4 mm;
1 kip = 4.448 kN;
1 psi = 0.006895 MPa;
1 kip/ft = 14.594 kN/m.

Discussion of this paper is invited. Please forward your comments to PCI Headquarters by July 1, 1981.

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