SEISMIC RESPONSE OF LARGE-PANEL STRUCTURES USING LIMITED-SLIP BOLTED JOINTS

A.S. Pall and C. Marsh

SYNOPSIS

Large-panel concrete construction offers many technical and economical advantages over conventional methods, but in seismic regions the construction of such structures is viewed with suspicion. This paper is devoted to the investigation of seismic response of panelized buildings using energy dissipating limited-slip bolted joints. Bolted joints have previously been proposed and used for large panel structures, basically, as an alternative to structural grouted joints for extending the benefits of industrialization to the erection procedure. The paper proposes the use of specified additional clearance in the slotted holes to provide a potential for a limited slip in the vertical joints. Under severe seismic excitations the limited slip in joints allows considerable energy dissipation without serious permanent deformations. This is a desirable mechanism for arresting severe seismic forces and can be a key factor in the survival of the structure.

Results of non-linear dynamic analysis indicate that the use of proposed limited-slip bolted joints, especially in vertical joints, can significantly improve the seismic performance of panelized buildings.

RESUME

Il existe dans l'industrie une inquiétude concernant le comportement sismique de structures fabriquées de grands panneaux. Une évaluation numérique d'une telle construction est faite. On démontre que des joints avec espaces libres, pour permettre un déplacement relatif dans les jointures verticales, aident grandement au comportement sismique, dû à la dissipation d'énergie qui se produit.

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INTRODUCTION

Large-panel (LP) construction is quite a popular form for multi-storey apartment buildings in the Soviet Union and Europe. Encouraged by the Operation Breakthrough, 1969, in the United States, interest is now steadily growing in LP construction in North America. The main hesitation in the adoption of such construction, in certain regions, has been the fear of their performance under severe seismic action. The fear stems from the fact that the development of flexural ductility, as available to other cast-in-place structures is more difficult to achieve in panelized construction. Ductility is considered to be an essential prerequisite for the survival of a structure in a major earthquake. The present seismic codes, which are based on the premise of ductility, lay penalties on structures not possessing adequate ductility and are not directly applicable to LP buildings.

On the other hand such structures have been constructed in earthquake zones in the Soviet Union, Rumania, Cuba, Japan and are gradually spreading to other places (1, 2, 3). The inspection of recent destructive earthquakes in Romania, Venezuela, and Soviet Union (1, 4) have provided enough evidence that LP buildings, designed for earthquake resistance, experienced minimum distress. While the other brick and framed buildings have failed or been severely damaged, the joints between the panels were the only vulnerable locations to develop cracks in the case of panelized buildings. LP structures are, therefore, capable of meeting the twin requirement of safety and damage control and are ideally suitable for apartment type buildings where 70-80% of the building cost is made up of non-structural elements.

The question arises as to how these buildings, in which the development of flexural ductility is limited, could perform so well in catastrophic earthquakes. It would appear that it is the overall energy dissipating capability of the structure which is the key factor for its survival. Earthquake damage in LP buildings has always been along joints with little damage in the panels. Cracking and slipping along these planes of weakness is associated with energy dissipation, a process similar to energy absorption during inelastic yielding in ductile structures.

In comparison to cast-in-place multi-storey shear walls, panelized walls are less rigid and therefore attract less inertial forces and also have a higher damping capacity. Quoting Despeyroux (5), "prefabricated walls are therefore to be considered better than monolithic walls as far as earthquakes are concerned". Muto's "slitted" shear walls (6), creating artificial vertical joints in the otherwise rigid walls is also based on the same philosophy.

ENERGY DISSIPATION MECHANISM

The joints are generally considered to be the weak links in LP buildings. Under severe ground motions, these natural planes of weakness are mainly responsible for introducing a non-linear behaviour in the overall building system, the panels themselves remaining in the elastic range. Thus, the connections are the only location of an energy dissipating mechanism. Hence, these very planes of weakness if properly harnessed, can be advantageously used for improving the seismic resistance of LP buildings.

The challange, therefore, lies in maximizing the energy dissipating mechanism of the joint. In LP construction, there are basically three locations for the joints. These are the joints between floor panels, the horizontal joints between wall panels, and the vertical joints between wall panels.

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

The horizontal joints between wall panels are not desirable locations for energy dissipation because sliding movements necessary for energy dissipation will cause inevitable destruction of the interfaces, and may render the structure unserviceable following the earthquake. A pure rocking type motion i.e. opening and closing of joint, in a typical horizontal connection, even if post-tensioned, does not cause energy dissipation (7, 21). Furthermore, the concentration of forces at the corners of the panels, associated with rocking motion may cause failure in either the connection or the panel (8).

Vertical joints between wall panels appear to be the most efficient source of energy dissipation. Unlike horizontal joints, the vertical joints, after the necessary slippage to dissipate energy, will come back almost to their original position. In North American systems, where large panel sizes are used, the possible location of vertical joints could be: (a) continuous joints in the end wall panels; (b) connections between corridor lintels; (c) at right angle joints between panels i.e. I or T sections, around elevator shafts and staircases, etc. However, most of the apartment buildings using the so called American system, do not have vertical joints and, therefore, lose the opportunity of energy dissipation through this mechanism. Uncoupled individual walls in such plans are also not desirable from the point of view of resistance against progressive collapse as these do not effectively provide an alternative path for load transfer in case of accidental failure of a wall panel. Nevertheless, even with the American system plans could be developed to incorporate vertical joints, by means of which a large portion of the seismic energy fed into the structure could be dissipated, thereby reducing seismic distress.

To maximize energy dissipation and satisfy the seismic demands, slipping vertical joints that possess "elasto-plastic" hysteretic characteristics provide a logical solution.

LIMITED-SLIP BOLTED JOINTS

The first use of bolted joints for LP structures was by Descon-Concordia of Montreal (9) in their projects for Operation Breakthrough in the United States. Basically, this patented system (22) was used to facilitate and speed up construction operations. Such joints were used for only slab to slab and horizontal wall to wall joints. Static tests to determine the strength of their prototype connection assemblies, built out of tubular steel sections having mill scale surface, have been performed by the U.S. National Bureau of Standards (10). No research work is so far reported on the performance of such connections when subjected to repeated reversals of loads.

Limited-slip bolted joints, hereafter referred to as LSB joints, consist of steel plates or sections with slotted holes which are friction bolted to steel inserts anchored into concrete panels. These are ideally suitable for the vertical joints and can also be used for all other slab and wall joints. Figs. 1 and 2 show the details for some of the wall to wall vertical joints. The length of the slot accomodates the normal fabrication and erection tolerances with an additional clearance to absorb energy by slipping. Provision of controlled slippage in the joint is the main feature of LSB joints. At the same time the slip is not to be so large as to distort the structure beyond acceptable limits.

With the LSB system, the horizontal joints in the wall will be grouted or dry packed after, say, three storey levels have been erected. The vertical joints are the ones expected to slip under severe seismic action, and these joints will not be grouted but sealed by other appropriate means. Floor joints will be grouted or otherwise sealed. The grouting process is independent of erection operation and can proceed uninterrupted, sheltered from weather.

Using this system, mechanization is extended to the jointing process so that the entire construction operation is totally industrialized. Fabrication of panels is already considered to be industrialized.

Several static and dynamic cyclic tests have been conducted on connection specimens having different faying surface treatments to evaluate basic design properties (11, 12). Load-deflection curves and hysteresis loops for wall connections, using 12.7 mm (1/2") dia. high strength bolts (ASTM A325), are shown in Figs. 3 and 4 respectively. In general, up to the point of slipping, the connection behaves elastically after which the slippage absorbs energy, simulating elasto-plastic behaviour. As the bolt reaches the end of the slot it goes into bearing, giving an ultimate load much higher than that causing slip. Appropriate surface finishes can be selected based on strength and economy considerations.

ANALYSIS

Non-linear behaviour is inevitable in LP structures when subjected to severe earthquakes. In order to investigate the participation of LSB joints in improving the seismic response of LP structures, typical apartment buildings (Fig. 5) of 10 and 15 storeys were studied. Analysis is made only for the exterior end walls having vertical joints. Structural idealization and properties of these walls and connections are shown in Fig. 6. Non-linear time history dynamic analysis was carried out using the computer program DRAIN-2D (13). Dynamic behaviour of the complete building is an extremely complex problem. To reduce the size of the problem, the following assumptions are made to isolate and examine the most important behavioural mechanisms:

(a) The floor diaphragms are infinitely rigid in their own planes and distribute the lateral forces on the building between different walls in proportion to their stiffnesses. The validity of this assumption has been questioned for low-rise panelized structures in which the stiffness of crosswalls is often as large as that of panelized floor diaphragams. Unemori (14) has suggested that for buildings, with 5 storeys or less the distribution of forces among the lateral walls may differ from that predicted under the rigid floor assumption by about 20-40% depending on the floor flexibility. The assumption of rigid diaphragms is however reasonably valid for crosswall buildings of more than 10 storeys. In this study the end wall, that was analyzed was assumed to carry approximately 1/4th the total lateral seismic force on the building.

(b) Vertical panel walls are considered as continuous elastic cantilevers. Although each cantilever wall includes horizontal joints, in the present analysis it it assumed that gravity loads (or post-tensioning if necessary for high sesismic accelerations) produce sufficient friction to prevent any shear slip or rocking.

(c) Mass and stiffness dependent type damping has been assumed corresponding to 5% critical damping.

(d) The foundations are rigid and soil structure interaction is ignored.

(e) The panels being large are assumed to remain in the elastic range. Non-linear behaviour is limited to the joints only.

(f) The stiffness of a connection is a function of the load on the connection and the relative displacement of the connected parts (Fig. 6).

Modeling Technique

The structure is idealized as an equivalent frame having wide columns with rigid arms and connected with LSB joints (Fig. 6a). The panels are modeled as beam-column elements. Flexural, axial and shear deformations are taken into account. Constant stiffness is specified as these elements remain in the elastic range.

There are two LSB connections per storey and these are considered as lumped at each floor level. These are modeled as axial elements yielding both in tension and compression to conform to the actual stiffness pattern of the connection shown in Fig. 6d. The third stage of the connection stiffness i.e. bolt coming into bearing, was not included in the present analysis in order to determine the maximum slippage requirements. This stage was, however, considered in the alternate approach described later. Hysteretic behaviour of the axial truss element, representing the connection, is shown in Fig. 6e. It should be noted that the average hysteresis loop includes degradation effect.

Translational displacements for all the nodes at a floor level are assigned identical values since the axial deformations in the floor diaphragms are assumed to be negligible. Also, due to the assumption that the rigid arms are of infinite stiffness and that the two panels are of equal dimensions, rotational displacements at all the nodes at a floor level are identical. This reduces the number of degrees of freedom and the problem is greatly simplified. Storey masses are assumed to be lumped at the column nodes only.

Seismic Input

The buildings are considered to be located in seismic zone 3 (15) which has a peak ground acceleration of 0.08g for a probability of one in 100 years and 0.25 g for a probability of one in 200 years. Probability of the latter earthquake during the life of the building is very low but the design can ensure the safety of the structure even in such extreme case.

For structures with a relatively high yield level, as is the case with LP structures, the peak accelerogram produces the most severe response (16). For this reason the N-S component of 1940 El Centro earthquake record, which is of the peaking type, was used. The input motion intensity was scaled to represent a peak ground acceleration of 0.25g. Response of the structure is also being evaluated by using an artificial earthquake record generated to match the Newmark-Blume-Kapur response spectrum.

To reduce computation time, the input, time-history of the earthquake was used for the first seven seconds only, followed by ficticious zero accelerations for the next two seconds to allow the structure to come to rest. Input ground motion history is shown in Fig. 11. The integration time step for the 10 storey building (period 0.3 sec) was kept 0.005 seconds and 0.01 seconds for the 15 storey building (period 0.6 sec).

Results of time-histories and maximum envelopes were obtained for displacements, moments, shears, axial forces etc. for panel elements and connections.

Approximate Approach for Non-Linear Response

Non-linear behaviour of panelized structures under severe seismic excitations can be approximated by a simple and practical method based on the Reserve Energy Technique. This alternative approach is much less complex than a non-linear time-history dynamic analysis which is valid for only one particular ground motion history, any change in parameters requiring an additional computer run. Such analysis may be justified for research or for large projects but it would be difficult to justify its use by the average practicing engineer. To compare the results the same buildings were analyzed by using the Reserve Energy Technique, originally developed by Blume (19), which was modified by the authors to suit panelized buildings (17, 18).

At peak seismic demands, feed-in kinetic energy less feedback from the structure into the soil layer, must balance elastic strain energy plus the energy lost in friction and the work done in non-linear deformations. Most of the present day multi-storey panelized buildings fall in the intermediate range of frequencies for which the principle of constant energy input applies i.e. the energy input elastically or inelastically remains the same. It is thus possible to calculate the kinetic energy fed into the building by using elastic reponse spectrum. The force-deflection diagram of a structure and that of connections are used as a measure of the total work capacity. The diagrams are plotted for some trial deflections until the energy demand and energy capacity of the structure are reconciled.

In this analysis two more assumptions were made; i) the connection were allowed to come into bearing i.e. the 3rd stage of connection stiffness, and ii) 10% energy feedback from the structure to the soil was allowed.

The results obtained from rigorous non-linear time-history analysis show close agreement with the approximate analysis.

DISCUSSION OF RESULTS

i) It is seen from the analysis for 10 and 15 storey buildings that both buildings are capable of resisting a seismic level of 0.08g within the elastic range. At higher ground accelerations the joints slip.

ii) Using coefficients given in the code, the quasi-static analysis yields shears which are very much less than those which would actually develop in these structures for a seismic level of 0.08g. This is so because for this seismic level the structures are still in the elastic range rather than in the inelastic range assumed in the code. However, the structure has sufficient resistance for the higher seismic forces.

iii) Due to the non-linearity introduced by LSB joints, the storey shears for a seismic level of 0.25g are approximately half of those obtained with elastic connections (bolted or welded) as shown in Figs. 7a and 7b.

iv) Maximum deflection envelopes for various seismic levels are shown in Figs. 9a and 9b. Close agreement is observed between the results obtained by the modified reserve energy technique and by the rigorous non-linear time-history analysis.

v) As the connections slip, redistribution of forces in the joints takes place until they become almost uniform throughout the height. Had the connections been elastic, the forces would be distributed as represented by the dotted lines shown in Figs. 8a and 8b. One of the effects of slipping joints is, therefore, to provide a limit to the load and to allow the capacities of all the standardized connections to be utilized.

vi) The energy dissipating LSB joints reduced the bending stress level in the panels. In the present example it is seen from Figs. 10a and 10b that the maximum compressive stress, for a seismic level of 0.25g, is reduced by nearly 40% from that which would have developed for elastic connections.

vii) Energy dissipation by the proposed joints, obtained by reserve energy analysis are shown in Figs. 14 and 15. It is seen that at higher seismic levels, nearly 25% of the total energy is dissipated by the joints in friction alone. Furthermore, the joints have contributed indirectly in softening the structure by introducing non-linearity to the otherwise elastic panels. Thus LSB joints, acting as friction dampers, are a primary source of energy dissipation.

viii) After the earthquake the stored elastic strain energy of the panels will restore the connections nearly to their original position and the building will be ready to face future earthquakes with the same efficiency. This is clearly seen from typical time-histories of panels and connections (Figs. 12 and 13).

ix) To dissipate energy the building must deform into the non-linear range, thus placing high ductility demand on the joints (20). This can be readily supplied by a slotted hole of any desired length. Limited slip i.e. allowing the bolt to come into bearing after a specified amount of slip, is desirable in controlling the deformations and forcing uniform ductility demand in all the connections. Fig. 16 shows the ductility demands in LSB joints. Ordinary grouted joints with reinforcing loops cannot meet such a demand without permanent damage. The term "ductility", although incorrect for the proposed joints is purposely used to be commonly understood. For the LSB joints it is the ratio of final displacement of the connection after slip to the elastic deformation before slip and is denoted as μ.

x) There is no yielding of material involved in the proposed joints and this is desirable from the point of view of damage or repairs.

CONCLUSION

Preliminary analysis shows that the use of the proposed energy dissipating LSB joints can significantly improve the seismic performance of large-panel buildings subjected to earthquakes. Furthermore, due to the softening of the structure caused by the slipping joints, the effective periods of vibration are increased which could be beneficial in attracting less seismic accelerations. In effect, the proposed joints act as structural dampers in arresting seismic forces.

That it be the vertical lines of connection in which these LSB joints are incorporated is of particular importance as 1) the level of energy dissipation is higher than with horizontal joints, 2) the joint strength can be uniform and all joints can contribute, 3) the building remains elastic and recovers with little or no permanent set, 4) the slipping of these joints acts as a safety valve limiting the load level exerted on the horizontal joints which will remain intact, 5) the building is softened without losing its elasticity and resilience, 6) the joints can be arranged to permit relative movement of the abutting panels without causing visible damage to the interior finish.

ACKNOWLEDGEMENTS

The research reported above was supported by the Natural Sciences and Engineering Research Council of Canada, and La Formation de Chercheure et d'Action Concertée du Québec. Their support is gratefully acknowledged.

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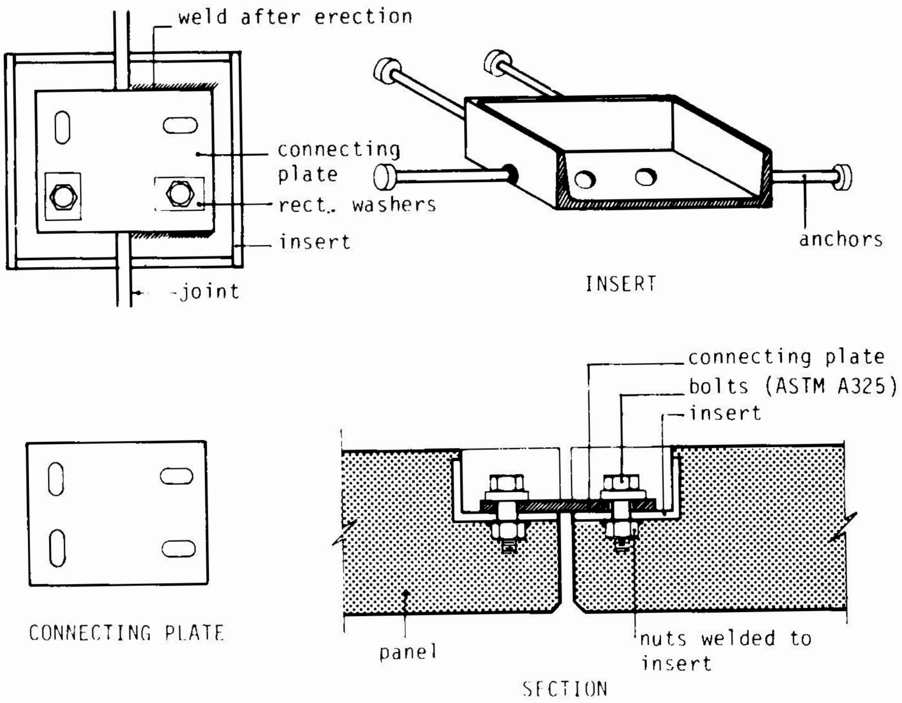


FIG. 1 - WALL-TO-WALL JOINT

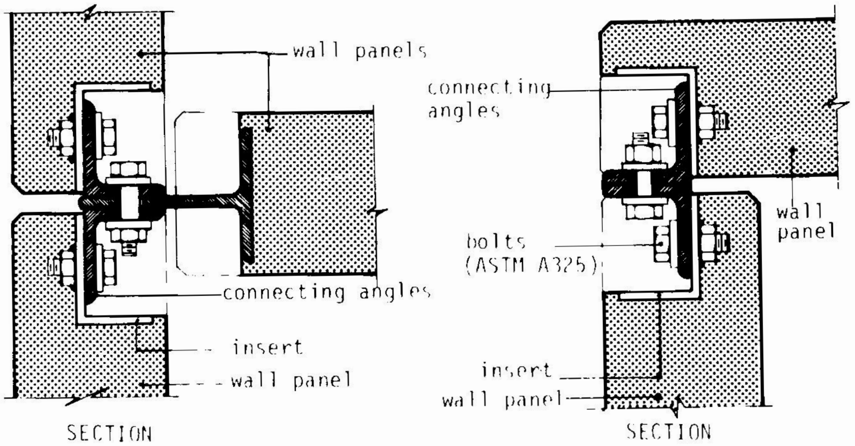


FIG. 2 - CORNER WALL-TO-WALL JOINTS

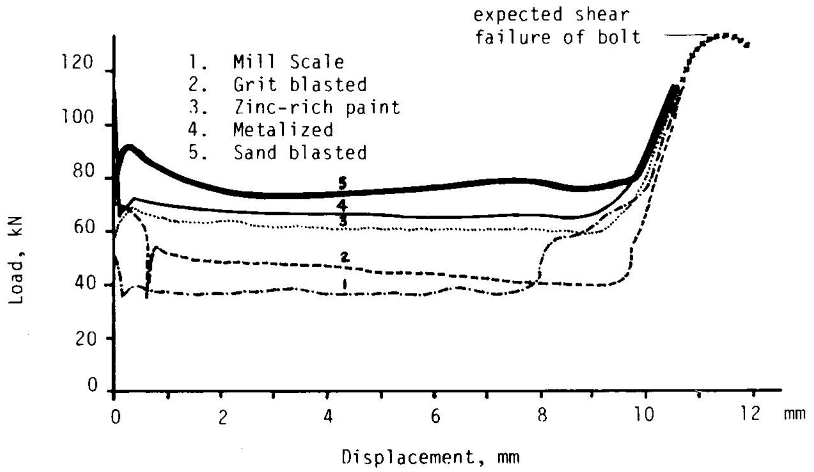
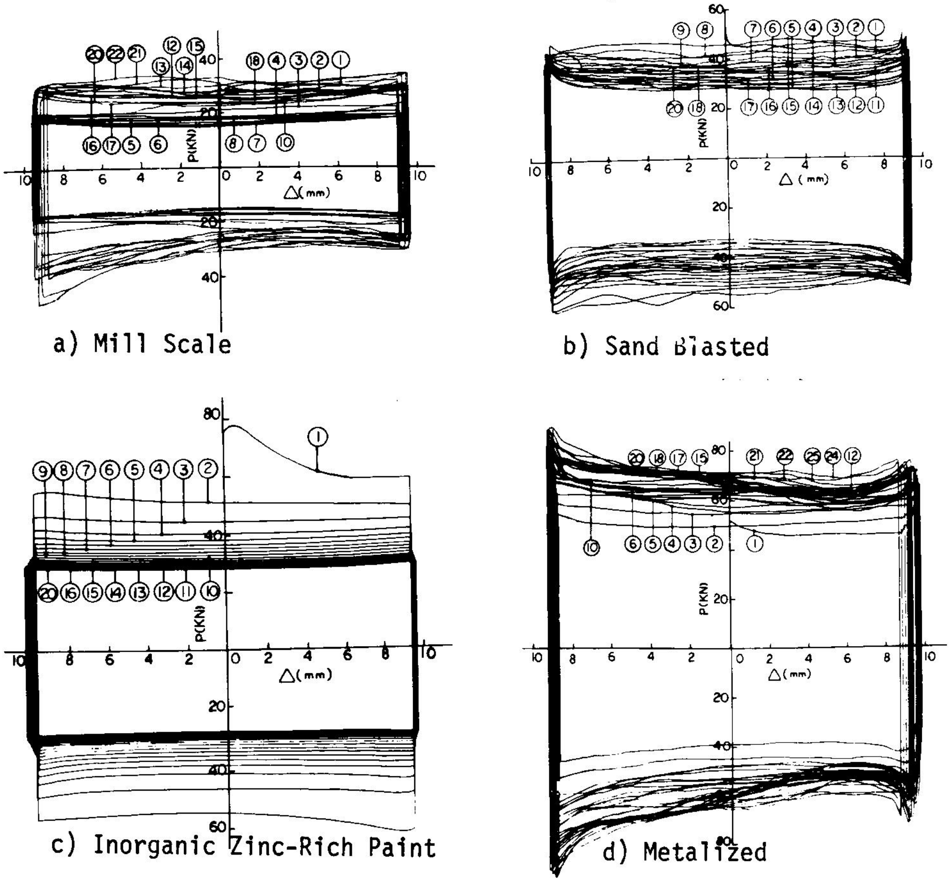


FIG. 3 - LOAD-DISPLACEMENT CURVE, WALL-TO-WALL JOINT



a) Mill Scale

b) Sand Blasted

c) Inorganic Zinc-Rich Paint

d) Metalized

FIG. 4 - HYSTERESIS LOOPS, WALL-TO-WALL JOINT

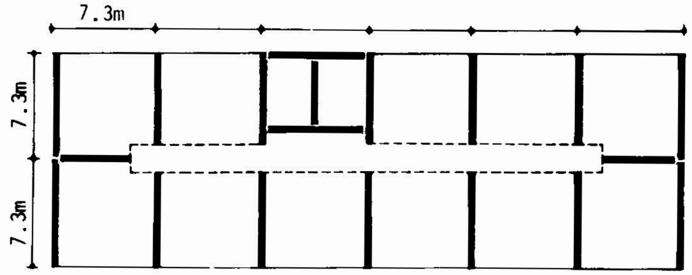
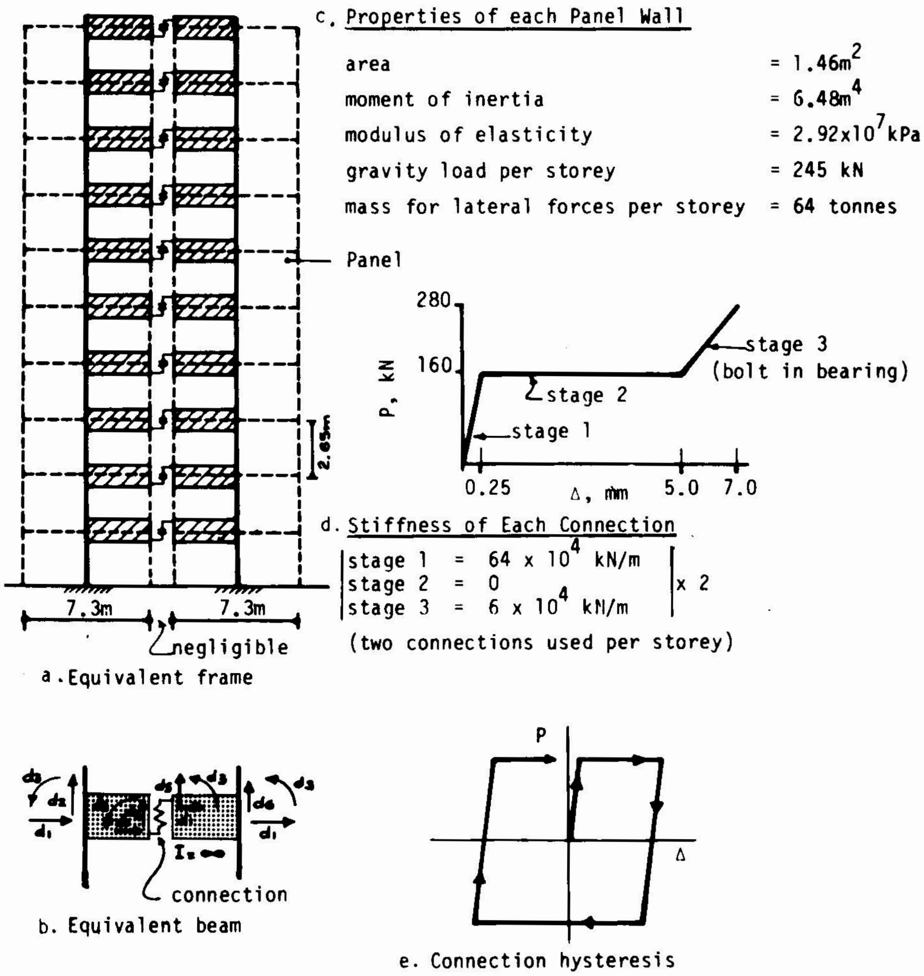


FIG. 5 - TYPICAL PANELIZED-APARTMENT BUILDING



a. Equivalent frame

c. Properties of each Panel Wall

area = $1.46m^2$
moment of inertia = $6.48m^4$
modulus of elasticity = $2.92x10^7$kPa
gravity load per storey = 245 kN
mass for lateral forces per storey = 64 tonnes

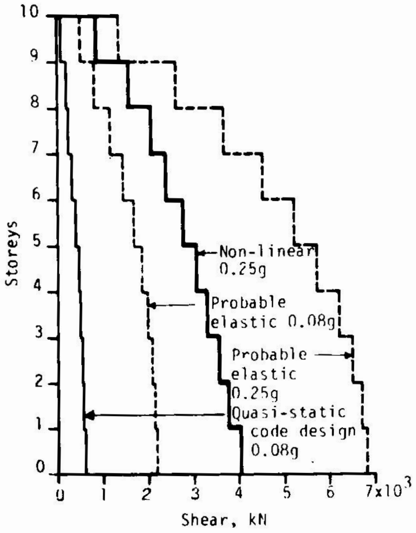
d. Stiffness of Each Connection

stage 1 = 64×10^4 kN/m
stage 2 = 0
stage 3 = 6×10^4 kN/m
x 2
(two connections used per storey)

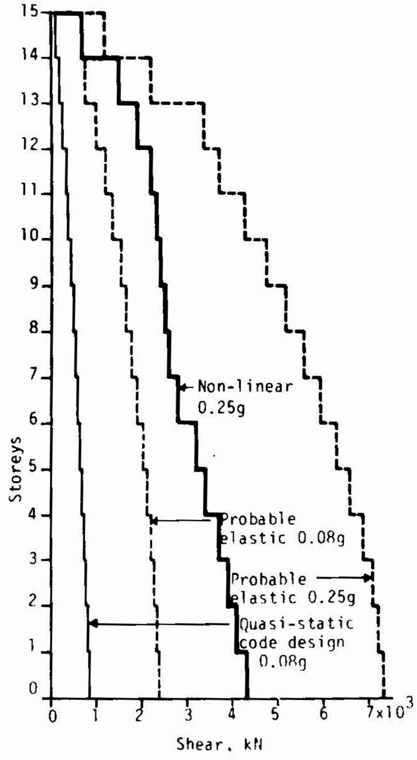
b. Equivalent beam

e. Connection hysteresis

FIG. 6 - STRUCTURE IDEALIZATION

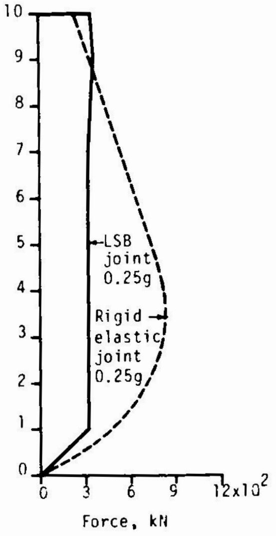


a. 10 storey building

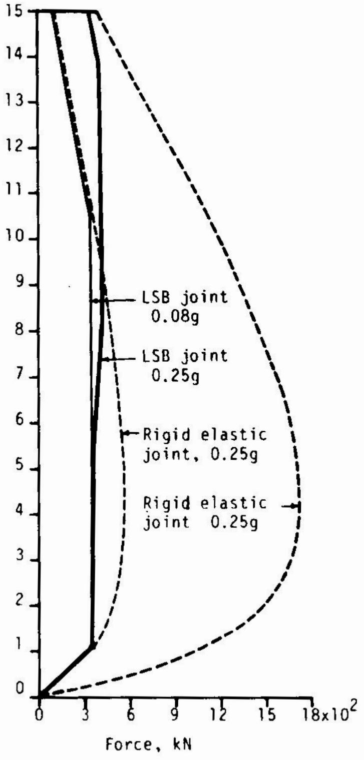


b. 15 storey building

FIG. 7 - STOREY SHEAR ENVELOPE

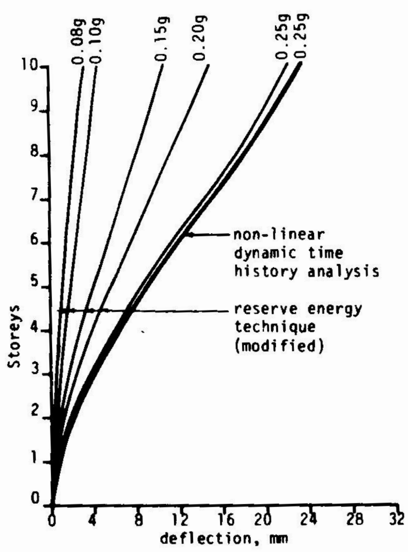


a. 10 storey building

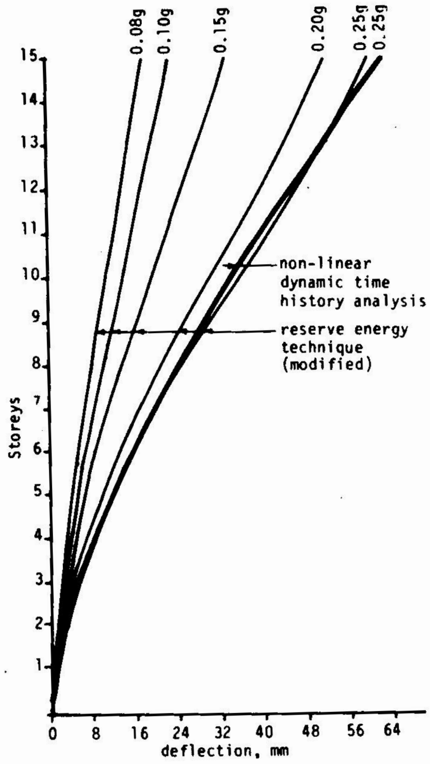


b. 15 storey building

FIG. 8 - FORCES IN JOINTS

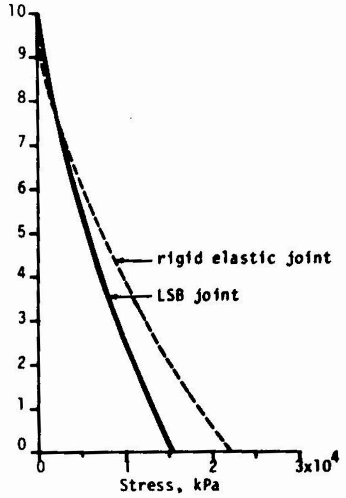


a. 10 storey building

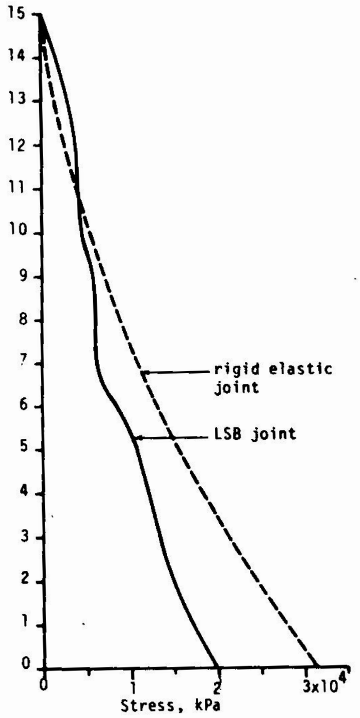


b. 15 storey building

FIG. 9 - DEFLECTION ENVELOPE



a. 10 storey building



b. 15 storey building

FIG. 10 - STRESSES IN PANELS (0.25g)

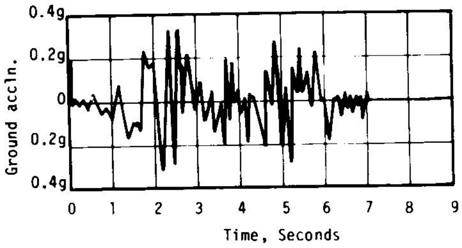


FIG. 11 - GROUND MOTION HISTORY
El Centro 1940 NS, first seven sec.

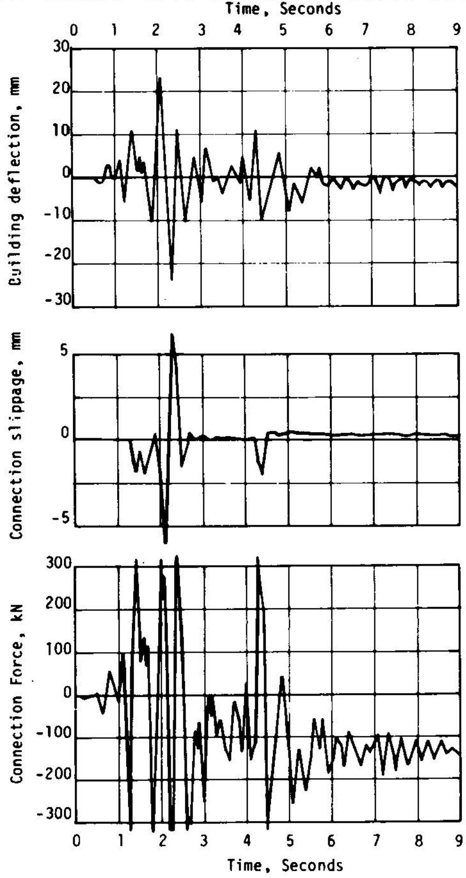


FIG. 12 - TYPICAL TIME HISTORIES
10 STOREY BUILDING

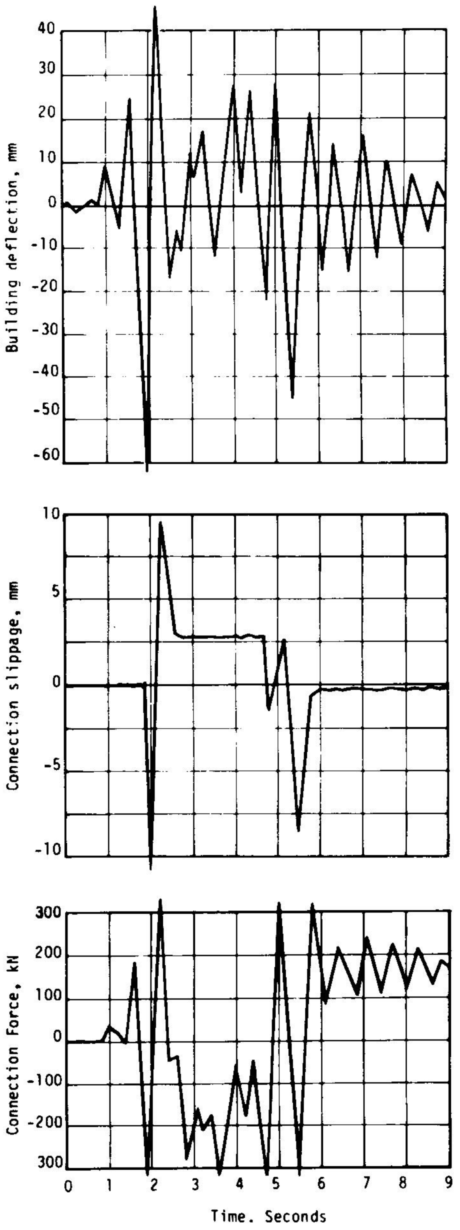


FIG. 13 - TYPICAL TIME HISTORIES
15 STOREY BUILDING

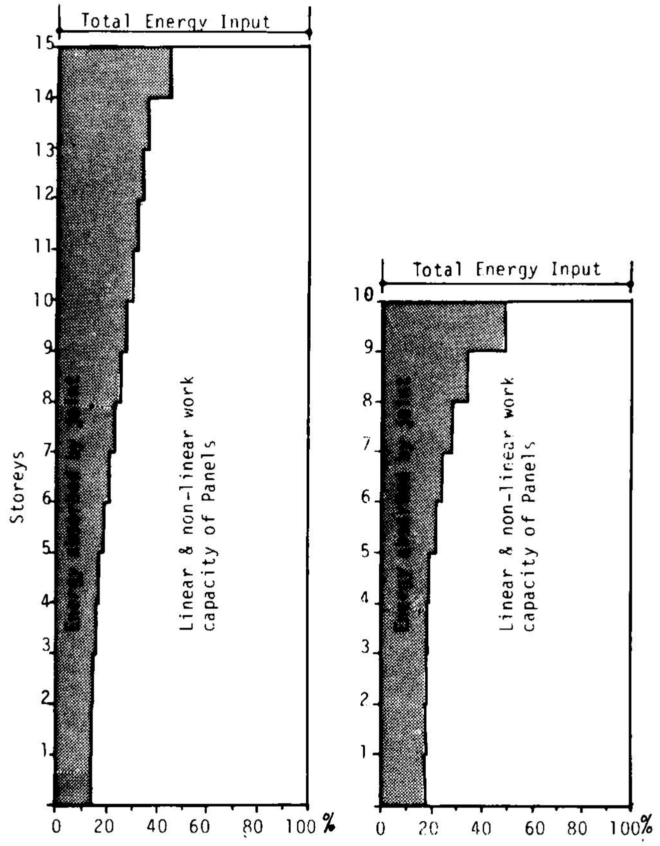


FIG. 14 - ENERGY DISSIPATED BY JOINTS AT DIFFERENT STOREY LEVELS (0.25g)

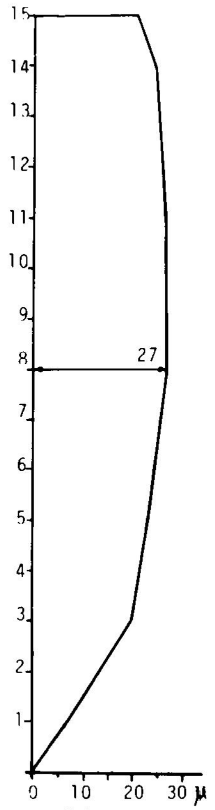
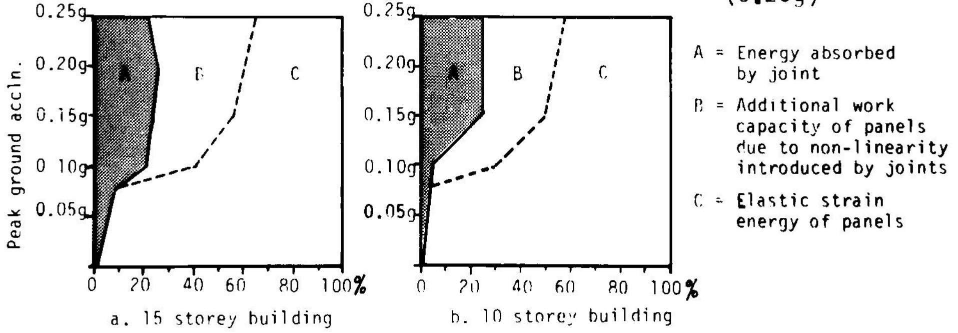


FIG. 16 - DUCTILITY DEMAND IN JOINTS, μ (0.25g)



a. 15 storey building

b. 10 storey building

FIG. 15 - TOTAL ENERGY DISSIPATED BY JOINTS