Paper No. 838. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

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DYNAMIC SEISMIC RESPONSE OF PRECAST PANEL STRUCTURES

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ABSTRACT

This paper deals with the dynamic analysis of a large multi-panel wall which is representative of the primary lateral load resisting component of a prefabricated building. The wall is modeled as a cantilever beam consisting of a series of elastic, plane-stress plates which have been stacked on top of one another. These plates are connected to one another along horizontal or vertical joints which constitute pre-defined planes of weakness. Within the joints the wall panels are interconnected by a series of nonlinear springs whose aggregate behavior is adjusted so as to mimic the way in which joints connecting them have been observed to behave during static load reversals. The computed overall response of the system was rather insensitive to the type of input ground motion utilized in the dynamic response computations, but was closely related to the peak aceleration of the input signal. It is concluded that design calculations must take into account the limited joint strength, and ensure that joints do not play the role of the weak link when the structure is subjected to earthquake ground motions. One way of ensuring this is to require reduced response modification factors to be used in design.

KEYWORDS

Structural engineering, precast large panel structure, earthquake response, horizontal joint, vertical joint, joint slip, elastic-plastic shear-slip relationship, tension-only spring, compression-only spring

INTRODUCTION

The pressing need to house rapidly large numbers of people in the post-Second World War period in Europe led to the development of large panel buildings. Strategically placed plants where quality control could be exercised, and where excessive skilled on-site labor was not needed were particularly appealing to centrally planned economic masterplans, and even today the tall, angular samples of this genre, divided into many small apartments for small families ring many cities. Large panel structures are now used less frequently for mass housing in western Europe and the US, and where they are utilized the choice is usually for hotels or office blocks because societal preferences do not favor the high-rise buildings comprising cellular flats with many tenants. Early examples suffered from serviceability deficiencies, such as improper sound, heat or moisture insulation, defective foundations leading to cracked joints. An economic height limit for this type of construction appears to be about 20 stories because the abundance of walls makes the weight per unit area rather heavy, leading to expensive foundations.

Many large panel buildings have been built in earthquake-prone areas of the world, but their performance record under the abnormal kind of loads represented by earthquakes has been less than exemplary, e.g., during the Taşkent earthquake of 1967 and in Bucharest in 1977. (Cast-in-place variants of large panel buildings, with abundant walls in both directions and poured in the so-called tunnel system have been built in great numbers in Turkey. To date, no major earthquake has affected these buildings.) Because of their size industrialized large panel structures have suffered from a general paucity of experimental research, particularly in relation to the way joints behave under severe external loads, such as during earthquake ground motions.

The primary aim of this paper is to present a set of results of dynamic response analysis on a particular structural configuration which is representative of a multi-panel shear walls. In the idealization, the panels are treated as linearly elastic, but they are connected to adjacent panels by means of discrete nonlinear springs which mimic the way such joints behave under static cyclic load reversals. The paper is not concerned with the design of joints, but examines the behavior of a solid shear wall, given the specific properties of joints in it.

JOINTS IN LARGE PANEL STRUCTURES

From the point of view of both analysis and design the most important area in a large panel structure is the joints. The structural system comprises floor slab planks and walls connected by horizontal joints which transfer the following primary actions:

(1) Vertical loads
(2) Horizontal shears
(3) Bending moments from the slabs

Many possible schemes for horizontal joints exist. Two of these are illustrated in Fig.1. From the seismic analysis viewpoint it is important to know how much shear can actually be transferred from one panel to those below and above it across the horizontal joints which enclose it. The primary mechanism of shear transfer is through friction. The frictional strength is essentially the coefficient of static friction μ multiplied by the cumulative normal compressive stress σ_n transferred by the panels above:

$$\tau = \mu \, \sigma_n \qquad (1)$$

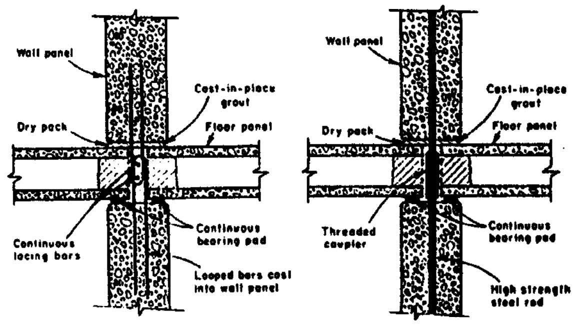

Fig. 1. Possible Horizontal Joints (Schultz, *et al.*, 1976)

The actual strength of a horizontal joint is arguably much more involved than the simple form given in Eq. (1) due to the continuity of the reinforcement in the vertical direction, or any transverse reinforcement which may be present, but these could be taken into consideration in an equivalent way by modifying the coefficient of friction. In Fig. 2 the cyclic load deformation curve for a horizontal joint tested with and without a normal stress is shown. It is immediately noted that the characteristic of these curves is similar to a rigid-plastic system if the initial effect of vertical joint reinforcement is neglected. Indeed, with increasing normal stress, the effect of the reinforcement is submerged within the frictional contribution to the strength (Schricker and Powell, 1980).

...d in different ways in the vertical direction. The connection may be in the form of

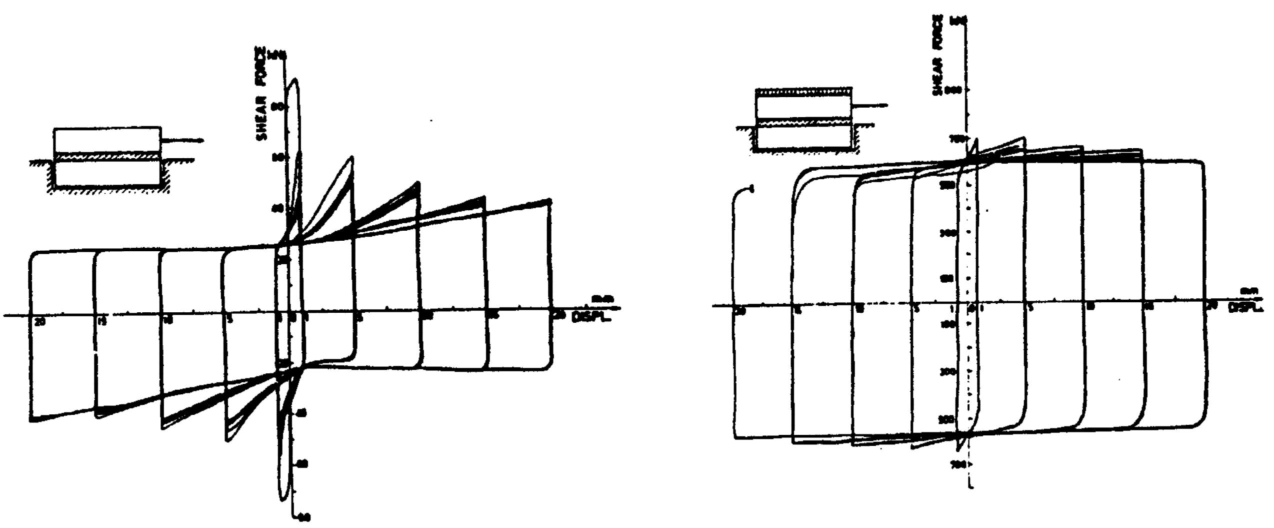

Fig. 2. Load-Deformation Curve for a Reinforced Horizontal Joint:
Effect of Normal Stress (Schricker and Powell, 1980)

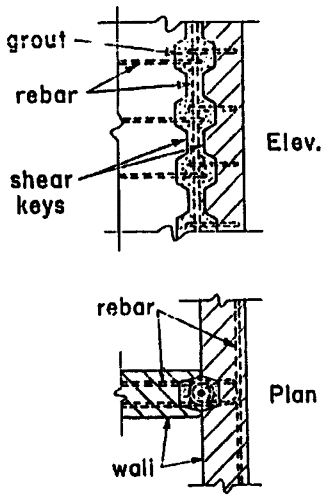

Fig. 3. Vertical Panel Edge Geometry and Connection (Becker and Llorente, 1978)

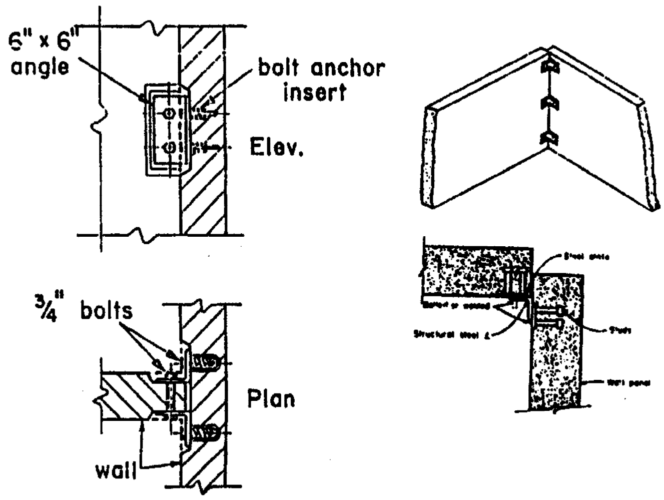

Fig. 4. Mechanical Connection in Vertical Joints (Mueller, 1981)

Under gravity loads horizontal joints are subjected primarily to normal stresses, but lateral earthquake loads produce complicated patterns of internal force distribution influenced by overturning moments, horizontal shears, uplift restraints, and interactions with the slabs. The main action to which the vertical joint is subjected is the vertical shear caused by bending, supplemented by the horizontal tie restraint which resists any tendency of the walls to act as independent vertical cantilevers. This restraint has a strongly nonsymmetrical character because separation of a panel from a neighboring one along the vertical direction is governed by the strength of the mechanical connectors, and is a possible deformation mode whereas any overlapping along the plane where they are in contact is prevented by the in-plane rigidity of the panels unless excessive dislodgment occurs. The strength of both the horizontal and the vertical joints must be such as to permit no crushing of the concrete or mortar along the contact planes (Caccese and Harris, 1987; Oliva *et al.*, 1990). Details of construction govern the way in which the wall will behave. Caccese and Harris (1987) reported that a combination of rocking and slip contributed to the wall deformations while Oliva *et al.* (1990) shear slip across the joints had very little effect on the behavior of the single walls, amounting to about 10 percent of the top displacement. Their tests

MODELING OF WALLS

From the structural engineering viewpoint a large panel building may be regarded as a series of essentially elastic (solid or pierced) wall panels interconnected at horizontal or vertical predetermined planes of weakness where the entire inelastic action, and a major part of the energy dissipation, occur during an earthquake. Modeling the modes of structural behavior across joints must mimic empirical data, but must also make allowance for gross simplifications, attempting to capture the essential tenor of overall response. In the finite element modeling of this study, the following considerations apply. Implicitly, these descriptions serve to explain the type of idealization adopted in this study.

The study reported in this paper is limited to a 10-story, two-bay large panel wall shown in Fig. 5. The size of each panel is 2.5x5 m, so that wall height is 25 m and width is 10 m. The most significant aspects of the way in which wall was modeled are encapsulated in Fig. 6 which portrays the rectangular wall panels connected by means of a series of translational springs, much like an interlocked mechanism of rigid blocks.

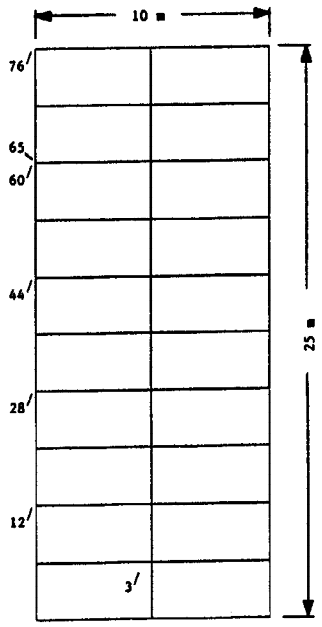

Fig. 5. Two-Bay Large Panel Wall

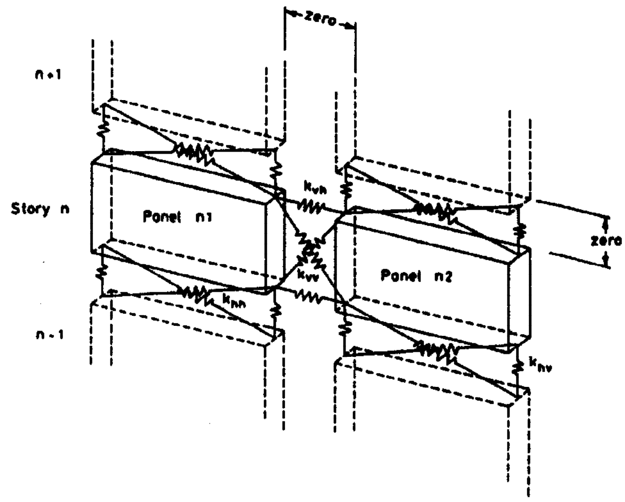

Fig. 6. Structural Idealization

The property of each connecting spring in Fig. 6 is represented by a generic symbol k with two subscripts, either h or v which stands for horizontal or vertical, respectively. The first subscript stands for the plane in which that spring is located, and the second denotes the direction in which it is effective. For example a spring designated as k_{hv} is located within a horizontal joint, but is effective when displacements at its ends are in the vertical direction. The springs work like gap elements, and their physical dimensions are immaterial.

Horizontal Joints

A gap element k_{hh} endowed with resistance against relative horizontal movement of the two panels along their common interface has a force-displacement relationship inspired by Eq. (1): this spring simulates dry friction with very high initial and unloading stiffness, but curtailed strength. Once the strength has been exhausted at the instant of slip the stiffness is zero. These springs are inserted also to mimic the response shown in Fig. 2, so that they are sensitive to the normal force on the joint. The coefficient of friction μ for joints is a highly variable quantity which is very dependent on the edge roughness of panels which may contain built-in asperities as well as dowel effects of any reinforcement. In the basic part of this study μ was assumed to be 0.5. The shearing force across the joint is carried by springs k_{hh} according to

$$F = k_{hh}\,\Delta \tag{2}$$

subject to $k_{hh} \to \infty$ but $|F| \le \mu N$ where N is the gravity force across a given horizontal joint.

A second feature is required to model appropriately the separation of panels from one another under overturning effects. The springs denoted with k_{hv} fulfil this role. In compression these springs possess "infinite" stiffness and strength so that the nodes they connect do not displace past one another, but separation is resisted by an elasto-plastic mechanism with finite stiffness and strength which is derived from a "wall boundary element" with 0.125% reinforcement ratio. In equation form:

$$F = k_{hv}\,\Delta \tag{3}$$

with the proviso $k_{hv} \to \infty$, with no limiting strength if $\Delta < 0$; $k_{hv} = 2.5e5$ kN/m if $0 \le \Delta \le 1.2e\text{-}3$ m and $F < 300$ kN, and $k_{hv} = 0.001$ of initial value otherwise.

Vertical Joints

Again, gap elements able to prevent overlapping of two neighboring panels, but permitting their separation are required. Springs denoted by k_{vh} which respond as described by Eq. (3), but with different properties simulating a given set of mechanical connectors served this purpose. Unlike horizontal joints normal stresses play a relatively minor role in these joints, so that relative shear slip can be accommodated with greater confidence through springs k_{vv} behaving as in Eq. (2). With reference to Fig. 6, the structural properties assumed for each set of elements are summarized in Table 1.

The entries for k_{hh} and k_{vv} in Table 1 reflect a simple design strategy: it is assumed that the wall is part of a structural system which has been designed for a base shear coefficient of 0.125. It is then possible to calculate the required horizontal and vertical joint capacities for a triangular distribution of the lateral forces by treating the wall as a vertical cantilever. These two sets of springs emulate the behavior in Fig. 2 and Eq. (2). The composite behavior of springs k_{hv} and k_{vh} is achieved by two collinear spring responding according to Eq. (3).

Table 1. Structural Properties

	Wall Panel		Horizontal Joint		Vertical Joint	
Story	E, kN/m²	t, m	k_{hh} (*)	k_{hv} (*)	k_{vv} (*)	k_{vh} (*)
1	2.5e7	0.2	1.25e7, 3000 e.p.	1.e12, 1.e12 c.o. 2.5e7, 1300 t.o.	5.0e7, 626 e.p.	1.e12, 1.e12 c.o. 6.25e5, 150 t.o.
2	"	"	1.25e7, 818 e.p.	1.e12, 1.e12 c.o. 2.5e5, 300 t.o.	5.0e7, 614 e.p.	"
3	"	"	1.25e7, 788 e.p.	"	5.0e7, 591 e.p.	"
4	"	"	1.25e7, 743 e.p.	"	5.0e7, 557 e.p.	"
5	"	"	1.25e7, 682 e.p.	"	5.0e7, 512 e.p.	"
6	"	"	1.25e7, 606 e.p.	"	5.0e7, 455 e.p.	"
7	"	"	1.25e7, 515 e.p.	"	5.0e7, 386 e.p.	"
8	"	"	1.25e7, 409 e.p.	"	5.0e7, 307 e.p.	"
9	"	"	1.25e7, 288 e.p.	"	5.0e7, 216 e.p.	"
10	"	"	1.25e7, 152 e.p.	"	5.0e7, 114 e.p.	"

(*) ... in these cells indicates the elastic stiffness in kN/m; the second entry is the strength in kN.

SELECTED RESULTS

The purposes of this work were twofold: (1) to examine the inelastic deformation of the structural configuration, and (2) to ascertain the extent to which forces and displacements were modified when joint slip and gap separation are controlled by inelastic springs with different strengths. A mass of 7 t was attached to each corner of each panel in Fig. 5 in addition to self weight. This corresponds to a tributary wall area of 60 m^2, and the total weight is 6650 kN. In Table 1 the value of k_{bh} at each story is adjusted proportionally to the weight of the mass above that story multiplied by $\mu = 0.5$. This value lends itself to a simple interpretation: the basic design strategy permits slippage to be initiated when the effective spectral acceleration above a given level equals 0.5 g, or when the design "base acceleration" has been amplified by a factor of four.

The dynamic response of the wall was performed with a general-purpose code (Anderheggen, *et al.*, 1985). The ground motion record utilized in the figures which follow is a 4 s long segment of the N05W component of the Parkfield earthquake of 27 June 1966 recorded at the Cholame-Shandon Array No. 5. This signal has a peak of 0.35 g. The first two computed elastic periods of vibration of the wall were 0.32 and 0.08 s, respectively. The viscous damping corresponded to 3.6 percent of critical in the first mode. The structure was first subjected to a constant vertical acceleration of 1 g before the ground motion reached the foundation.

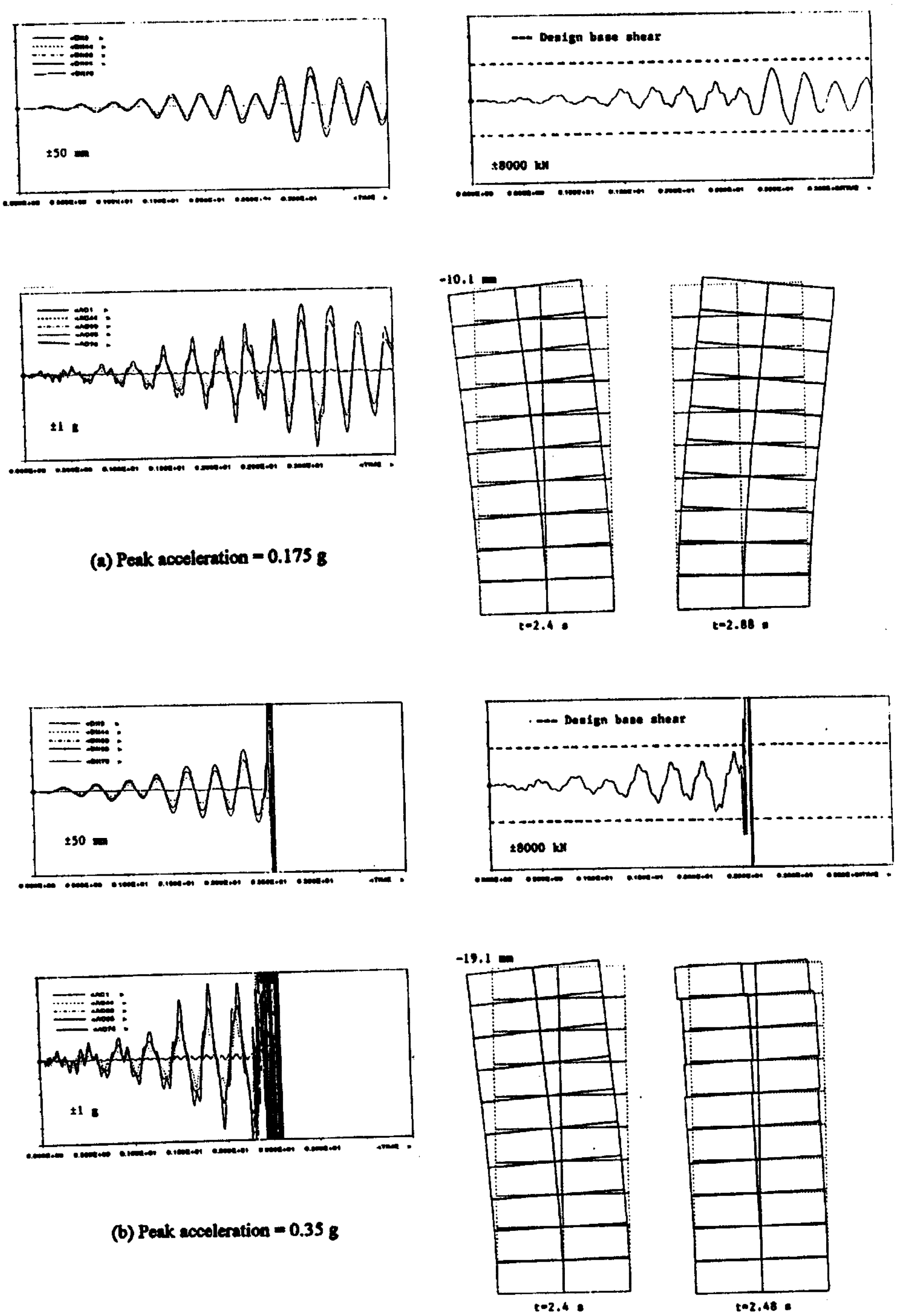

Fig. 7. Dynamic Response for Unmodified Joint Strengths

In Figs. 7 and 8, the horizontal displacements of points highlighted in Fig. 5 (and entered into the corresponding legend), base shear and relative acceleration time signals are shown along with instantenous deformed geometry snapshots. Figure 7 is for the joint strengths in Table 1, and Fig. 8 for twice these values. In Fig. 7(a), the input signal has been scaled by 0.5, in Figs. 7(b) and 8(a) by 1.0, and in Fig. 8(b), by 1.5. For time signals the upper and lower limits of the vertical axis are inserted on the diagram itself. To enable easy visual comparison, each deformed shape figure has the top displacement (joint 76 in Fig. 5) entered at t=2.4 s. The design base shear has also been superposed on the corresponding frame of each figure.

It is noted that for base acceleration peaks of 0.175, 0.35, and 0.525 g, the top displacements at t=2.4 s are in ratios of 1:2:3, respectively. For peak ground acceleration equal to 0.175 g, and joint strength ratio of 1 (Fig. 7(a)), or for peak acceleration of 0.35 g and strength ratio of 2 (Fig. 8(a)) when the design base shear is not surpassed, the response is linear with very small top drift. These figures are in fact almost exact replicas of one another in ratio of 1:2. If either the input motion is too strong, or the joint strength is exhausted, then there is a rapid displodgment process which quickly leads to a numerical failure, as illustrated by the terminated response time signals, or the jumbled geometrical arrangement of the large panel blocks in Figs. 7(b) and 8(b). Once the numerical disintegration was initiated, it was not possible to recover numerical stability in the response calculations, and the process had to be abandoned. Panel stresses were much smaller than typical concrete strengths in industrialized construction.

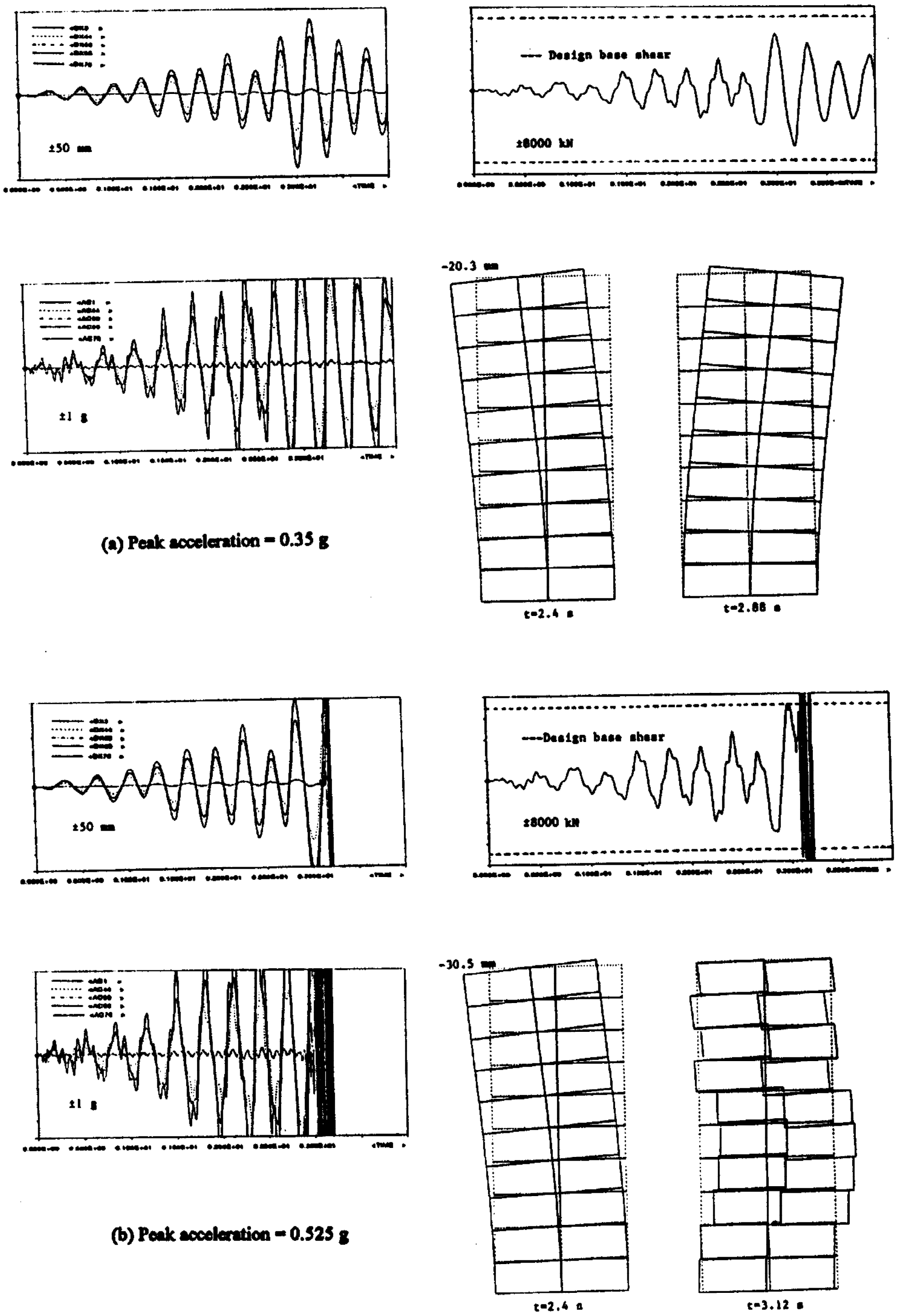

Fig. 8. Dynamic Response for Modified Joint Strengths

It would be premature to state that slippage (mostly across horizontal joints) immediately portends some form of progressive structural failure because, in reality, grinding of the panels in the joints is a good mechanism of energy dissipation, and small amounts of dislodgment may not necessarily be a cause for objection from the safety point of view. This would be a cause for concern from the serviceability viewpoint because distorted ... tens of mm in width, would be very difficult to remedy later.

An interesting set of results belongs to the relative acceleration in the two basic cases. The sharp high frequency peaks are associated with gap opening and closing in the joints. When joint behavior is still within the elastic limits, system response is elastic, and occurs at the fundamental period of 0.32 s. With the entire cross section of the wall contributing to the bending stiffness, displacements increase linearly from the base, and higher mode response is virtually nonexistent, as evidenced by the snapshots at 2.4 s. When the limiting base shear, derived from the springs k_{hh} at the foundation interface, is exceeded sudden slippage ensues.

CONCLUSIONS

The principal conclusions of this study are:

(1) The computed response of prefabricated large panel shear walls is largely dependent on the horizontal and vertical joint strength and their post-yield behavior.

(2) Because of the great initial stiffness and small period, dynamic response is dependent on the peak ground acceleration, and is possibly oblivious to the type of the input motion.

(3) An admissible design strategy should ensure that joints remain within the elastic range by requiring variable response modification factors in design, i.e. smaller factors for the joints and larger ones for the structural system as in bridge connections.

ACKNOWLEDGMENTS

The calculations for structural response reported in this paper were performed at the Institute of Structural Engineering (IFBK) of the Swiss Federal Institute of Technology in Zürich (ETHZ). I am grateful to Professor E. Anderheggen for enabling me to have remote access to the computer on site and to Mr. D. Ekchian for his assistance.

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