SEISMIC CHARACTERISTICS OF COMPOSITE PRECAST WALLS

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SYNOPSIS

The paper explores the role of vertical connections in the seismic response of composite walls used in large panel precast concrete buildings. On the basis of this examination a strong horizontal joint, weak vertical joint aseismic design philosophy is suggested.

The shear medium theory or continuous medium method is briefly reviewed as it applies to the coupling phenomenon in precast walls. A simple explicit formula for the fundamental period of composite precast walls and coupled shear walls is presented.

With the linear elastic characteristics as a basis, the effect of vertical connection stiffness, strength and cyclic degradation on the inelastic seismic behavior of composite precast walls is discussed. This discussion along with the proposed design philosophy is illustrated with a series of computer results. These results indicate that if vertical connections can be developed that exhibit a stable elasto-plastic hysteretic behavior, the walls and the vulnerable horizontal connections can be efficiently protected by deliberately designing weak vertical joints. With the actual behavior of presently used connections in mind, the need for the development of new vertical joints is pointed out and some promising approaches are mentioned.

RESUME

Cette communication explore le rôle des joints verticaux lors du calcul sismique des panneaux préfabriqués en béton. Une philosophie de base est donc suggérée où les joints horizontaux possèdent une résistance adéquate mais les joints verticaux sont délibérément faibles.

A partir des caractéristiques linéaires et élastiques, l'effet vertical des joints du point de vue rigidité, résistance, dégradation cyclique est étudiée afin de connaître le comportement non-linéaire des murs préfabriqués. Cette discussion basée sur la philosophie de design est illustrée à l'aide de plusieurs résultats numériques. Les résultats démontrent que si le joint vertical est capable d'assumer un comportement élasto-plastique stable, les murs et les joints horizontaux, qui sont plus vulnérables, peuvent être protégés d'une façon adéquate à la condition de rendre les joints verticaux faibles.

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INTRODUCTION

The extended use of large panel precast concrete buildings (see Fig. 1) in many of the seismic regions of the world, raises new questions in aseismic design. These questions center mainly around the connections used in panelized buildings (see Fig. 2) (1,2). The design of the connection details involves the balancing of economics and ease of construction on the one hand with the need of creating continuity on the other. This results normally in a system, in which the connections represent the weakest structural element with respect to both stiffness and strength. Connections are therefore not merely secondary elements that can be designed on the basis of an overall structural analysis, rather they represent one of the major factors governing the overall seismic response of the structure.

This paper explores the role of vertical connections in aseismic design of panelized walls that have been joined together to form composite walls with planar, I, U, T or box-type sections (see Fig. 3). The behavior of horizontal joints has been investigated elsewhere (3,4) and is treated only when relevant in the present context.

In non-seismic regions the tendency has been to design vertical joints capable of developing the full monolithic strength of composite walls. Typical of this approach are the preliminary recommendations by Hansen, et al (5) for an 'elastic limit' design for composite walls. In aseismic design situations the problem of vertical joint design should be viewed from a different perspective. Under severe earthquake excitation inelastic action will in all likehood occur and most probably it will occur in the critical connection regions. Directing the primary inelastic action to structural elements which exhibit favorable hysteretic behavior and do not threaten the overall stability when degrading then becomes a major aseismic design issue. In this manner, the vertical connection may represent an important opportunity in the aseismic design of large panel buildings and it is to this idea that the remainder of this paper is addressed.

ELASTIC DYNAMIC PROPERTIES OF COMPOSITE PRECAST WALLS

The basis for the understanding of the inelastic behavior of a structure requires a thorough knowledge of the main characteristics and governing parameters of its elastic response. The shear medium theory provides such a basis for understanding the response of panelized walls coupled through vertical connections and is shortly reviewed. As a by-product, a relatively simple explicit expression for the fundamental period of composite walls is presented.

Shear Medium Theory

The shear medium theory has been widely used in the area of coupled shear walls and shear wall buildings. It allows not only a straightforward analysis of the overall response of the classical case of two shear walls coupled by coupling girders, but has also been successfully used in computer programs for complex shear wall buildings to reduce the number of degrees of freedom involved. The equations for two or three symmetrically coupled shear walls are essentially governed by only two parameters. These parameters represent therefore an ideal criterion to classify coupled shear walls. The contributions to the subject in the literature are numerous for both the static (6,7,8, 9,10,11,12) as well as for the dynamic (13,14,15,16,17,18) cases.

The theory is equally applicable to precast concrete panel buildings with vertical joints and in reality more accurate. For grouted vertical joints the basic simplification of the theory, the replacement of the discrete coupling girders by a continuous shear medium, is exact. For connections with discrete mechanical connectors the error involved is smaller, because the spacing of the connectors is closer than that of the coupling girders. Moreover the problems with the flexible encasement of the coupling girders vanish. The theory is particularly simple for structures with constant cross-sectional properties and constant coupling stiffness. This is usually the case in panelized buildings and will be assumed throughout the paper.

For the derivation of the differential equation the reader is referred to the above cited literature. To establish the basis for the remainder of the paper, the governing parameters and the differential equation are simply restated in a form suited for panelized buildings. The basic case of two unequal coupled walls shown in Fig. 4 covers a great part of the other configurations found in lateral force resisting elements of panelized buildings: three coplanar, symmetrically coupled walls and composite walls with symmetric U, I or double-symmetric box-type shapes.

The basic assumptions for the shear medium theory are:

- Discrete coupling elements are evenly smeared over the joint length.
- The lateral deflections of the individual walls are equal.
- For the individual walls classical beam theory holds, in particular the assumption that plane cross-sections remain plane.

The second assumption is somewhat doubtful for two unequal planar coupled walls, particularly if the relatively flexible mechanical connectors are used. It is surely accurate for the case of flange walls coupled with web walls (Fig. 3d). With the notation and conventions given in Fig. 4 the differential equation for the shear flow q is

$$q'' - \alpha^2 q = - \alpha^2 q_\infty \tag{1}$$

$$\alpha^2 = \beta/\gamma \tag{2a}$$

$$\beta = \frac{kc^2H^2}{EI_0} \tag{2b}$$

$$\gamma = 1 - I_0/I_\infty \tag{2c}$$

where primes denote derivatives with respect to the dimensionless coordinate $\xi = x/H$. It is significant to note that Eqs.(1) and (2) and all definitions given below apply equally to both the two-wall configurations shown in Fig. 3a and b and the symmetric three-wall configurations (Fig. 3c and d), if in the latter cases k, q and q_y denote the sum of the stiffness, shear flow and yield strength of the (equal) connections.

The term c denotes the distance between the centers of gravity of the walls adjoining a connection. I_0 is the sum of the moments of inertia of the individual walls, $I_0 = \Sigma I_i$, i.e., the effective moment of inertia of the uncoupled, but equally deflecting walls (k = 0). I_∞ is the moment of inertia of the rigidly coupled walls acting as one integral beam ($k = \infty$, plane sections remain plane). The shear flow that would occur in the integral beam at the location of the connection is given by the well-known expression VQ/I_∞, where Q is the statical moment of the area separated by the connection with respect to the center of gravity of the integral cross-section. But it can be easily shown that

$$\gamma = n\frac{cQ}{I_\infty} \tag{3}$$

where n is the number of connections. Hence, in agreement with the aforementioned conventions, the shear flow at the connection, or the sum of the shear flow at the connections, respectively, occurring in the integral beam can be generally written

$$q_\infty = \gamma \frac{V}{c} \tag{4}$$

Introducing Eq. (4) in Eq. (1) results in the form of the differential equation usually encountered in the literature

$$q'' - \alpha^2 q = - \alpha^2 \gamma \frac{V}{c} = - \beta \frac{V}{c} \tag{5}$$

Also, the γ parameter is usually defined in a form similar to

$$\frac{1}{\gamma} = 1 + \frac{I_0}{c^2} \frac{A}{A_1 A_2} \quad , \; A = A_1 + A_2 \tag{6}$$

However, this definition applies only to the two-wall configuration shown in Fig. 4.

The form of Eq. (1) immediately shows that the stress distribution in the coupled walls is defined by the stress distribution according to integral beam theory and the parameter α. It is significant to note, that for a constant or linear shear force and hence shear flow, i.e., a cantilever with a point load at the top or a constant load distribution, the integral beam theory solution satisfies Eq. (1). Hence the differences between integral beam theory and shear medium theory result only from differently specified boundary conditions. This points to the possible importance of the actual boundary conditions. In the classical case of a fixed-end cantilever, the boundary conditions of zero slope at the bottom and zero moment and normal force at the top translate into

$$q(\xi = 0) = 0 \tag{7a}$$

$$q'(\xi = 1) = 0 \tag{7b}$$

Different boundary conditions are treated in the literature (7).

Fig. 5 shows the shear flow distribution along a connection and the ratio, K_4 (see Eq. (12b)), of the coupled and uncoupled top deflections as functions of the parameters α and γ for a triangular load distribution. The curves are adapted from the work of Coull and Choudhury (9).

The shear medium parameters being an ideal criterion for the classification of coupling problems independently of the actual method of analysis used, their meaning shall be shortly restated. Note that they are, contrary to other definitions in the literature, dimensionless.

α^2 is the relative coupling stiffness. Physically it can be interpreted as the ratio of the vertical relative displacement between the walls and between the two edges of a joint for unit forces acting on top of the walls and on the entire connection at the wall-connection interfaces (see Fig. 6). The overall stiffness and hence deflections of a structure are very sensitive to α for low and nearly insensitive for high values of α (Fig. 5b). The latter is even more evident for the fundamental periods of coupled walls (see next section). For a triangular and, hence, approximate seismic load distribution, the connection shear flow is relatively even in the sensitive, low α-value range, whereas it approaches the uneven distribution of an integral beam for high α values in the insensitive range. In the insensitive range the structure behaves nearly monolithic. Note that α is proportional to the height of a structure but only proportional to the square root of the connection stiffness. Thus the height of a building has a much stronger influence on the degree of coupling than the connection stiffness itself.

γ is a measure of the relative difference in stiffness and deflections between the uncoupled and the rigidly coupled systems (Eq. 2c). It thus defines the stiffness range over which the overall stiffness can be influenced by the selection of the connection stiffness. In view of Eq. (6) it is often interpreted as representing the effect of the axial deformations of the individual walls. Neglecting axial deformations results in $\gamma = 1$. However, the more general definition Eq. (2c) shows that such an approximation can never converge to the integral beam solution for $\alpha \to \infty$ and as shown by Fig. 5b the error involved may be large.

Integration of equation (4) gives

$$T_\infty = \int_x^H q_\infty dx = \gamma \frac{M}{c}$$

$$T_\infty c = \gamma M \qquad (8)$$

where T is the normal force in the walls due to coupling. Hence from the point of view of forces, γ gives directly the fraction of the total overturning moment that is resisted by the axial couple in the rigidly coupled system ($k = \infty$). Surely then, when illustrating the effectiveness of coupling of a system, the axial couple should be compared with this fraction of the overturning moment rather than with the total overturning moment.

Finally then the parameter β is merely a convenient dummy parameter to calculate α with no physical meaning except that $\beta = \alpha^2$ for the unrealistic case $\gamma = 1$.

It remains to discuss the values of α and γ found in panelized buildings. The estimated values refer only to coupling through vertical connections. For coupling through coupling girders, lintels or floor slabs, the literature on coupled shear walls applies.

Because in walls coupled by vertical joints the panels stand directly side by side, the distance between the centers of gravity is fixed and cannot be used to influence the degree of coupling. For the same reason the γ values are generally lower than in shear walls coupled by coupling beams. For a U shape with equal sides, $\gamma = 0.5$. For two coplanar coupled walls, for a box shape with equal sides or an I shape with a flange width of half the web depth, $\gamma = 0.75$.

The relative coupling stiffness α found in panelized buildings seems to cover the full range of coupling degrees. In the computer study reported below, a U-shaped composite wall of a ten-story building with dimensions usually found in large panel precast concrete building had a coupling stiffness $\alpha = 4$. The value was derived assuming two studded mechanical connectors per story and using stiffness values in the range of reported test results. For a box-shape with similar dimensions and door openings $\alpha = 5$ was found. Because α is proportional to the height, α values of 2.0 and 2.5 for five-story and 6.0 and 7.5 for the fifteen-story building with the same cross-section and connection properties are indicated.

For grouted reinforced keyed or plain joints, Hansen et al. (5) suggested a value of 300 kg/cm^3 as a reasonable lower bound stiffness to be used in elastic limit design for normal stresses. Depending on the assumed effective joint width, the above α-values correspond approximately to 90 - 130 kg/cm^3. Hence, for the same building the estimated lower bound α-values for grouted joints are roughly higher by a factor $\sqrt{3}$ than for the investigated dry joints, i.e., in the order of magnitude of α = 4, 8 and 12 for the five, ten and fifteen story building, respectively. Note however that values of up to 1500 kg/cm^3 and more have been reported for the stiffness of grouted keyed joints.

Fundamental Period of Composite Precast Walls

It is well known that the shear medium theory leads to a 6th order differential equation of motion for coupled shear walls. A simple closed form expression for the fundamental period is therefore not likely. By setting $\gamma = 1$, $\alpha^2 = \beta$, a 4th order differential equation of motion results which is similar in form to the equation of motion of a bending beam under axial tension or to a bending beam coupled with a shear beam. However, in view of the low γ values in panelized buildings, setting $\gamma = 1$ results in significant errors.

The problem has been solved in various ways. On the one hand the exact eigen value problem has been solved numerically for the fundamental period (14) as well as for the higher periods (15). Both studies treat the general case of unequal lateral deflections of the coupled walls and in the latter the vertical inertia forces are also included. On the other hand, various approximate solutions based on energy methods have been reported (13, 16, 17). To the authors' knowledge, however, none has resulted in a relatively simple, explicit closed form expression.

Such a solution shall shortly be presented. It is based on an energy method, more specifically, the improved Raleigh Method (18). In view of the fact that the expressions for the deflected shape of coupled shear walls are relatively complicated, a frequency expression is desirable in which the deflected shape is not squared.

The intermediate frequency expression resulting from one Stodola-Vianello iteration step satisfies this requirement.

$$\omega_{o1}^2 = \frac{\int_o^H m\, z_o^2\, dx}{\int_o^H m\, z_o\, z_1\, dx} \qquad (9)$$

In Eq. (9) m denotes the mass per unit length; z_o is the first assumed mode shape or inertia force distribution; z_1 is the derived deflected shape for the distributed load $p = m\, z_o$ and satisfies the governing differential equation and all boundary conditions.

Eq. (9) results from equating the maximum kinetic energy calculated on the basis of the initial assumed shape with the maximum strain energy calculated with the derived shape. In the standard Raleigh frequency expression the assumed shape z_o must satisfy the geometric boundary conditions and certain continuity requirements. However, using Eq. (9), the requirements with respect to continuity and satisfaction of geometric boundary conditions are less stringent (19). More specifically, for a simple cantilever beam a triangular z_o is also feasible. It can be mathematically shown that the resulting frequency is still an upper bound. For the simple cantilever beam with bending stiffness EI, mass per unit length m and height H, the evaluation of Eq. (9) gives

$$\omega = \sqrt{\frac{140}{11}\,\frac{EI}{mH^4}} = 3.57\sqrt{\frac{EI}{mH^4}}$$

for a triangular assumed shape z_o. The constant 3.57 differs only by 1.5% from the exact value $(1.875)^2$.

However, the simple cantilever response is the limiting case for the coupled wall structures shown in Fig. 3 for both zero and infinite connection stiffness. Thus it may be expected that Eq. (9), together with a triangular assumed shape, will also give acceptable accuracy for the intermediate range of coupling stiffness.

Referring again to Fig. 4 for notation and conventions, the deflected shape of a coupled shear wall for the triangular load distribution

$$p = m\,z_o$$

$$z_o = 1 - \bar{\xi} \tag{10}$$

is given by

$$z_1 = w = Z_1 \cdot \psi_1(\bar{\xi}) \tag{11a}$$

where

$$Z_1 = \frac{11}{120}\,\frac{2}{7}\,\frac{mH^4}{EI_o} \tag{11b}$$

$$\begin{aligned}\psi_1 = {} & \frac{7}{2}(1-\gamma)\left[-\frac{1}{11}\bar{\xi}^5 + \frac{5}{11}\bar{\xi}^4 - \frac{15}{11}\bar{\xi} + 1\right] + \\ & + \gamma\frac{420}{11}\left[\frac{1}{\alpha^4}\cosh\alpha\bar{\xi} - \frac{\sinh\alpha - (\frac{\alpha}{2} - \frac{1}{\alpha})}{\alpha^4\cosh\alpha}\sinh\alpha\bar{\xi}\right. \\ & - \frac{1}{\alpha^4}\left(\cosh\alpha - \tanh\alpha\left(\sinh\alpha - (\frac{\alpha}{2} - \frac{1}{\alpha})\right)\right) \\ & \left. + \frac{1}{\alpha^3}(1-\bar{\xi})(\frac{\alpha}{2} - \frac{1}{\alpha}) + \frac{1}{\alpha^2}\,\frac{1}{6}(\bar{\xi}^3 - 3\bar{\xi}^2 + 3\bar{\xi} - 1)\right]\end{aligned} \tag{11c}$$

The terms α and γ are defined by Eqs. (2). For the top deflection there results

$$w(\bar{\xi} = 0) = \frac{11}{120}\,\frac{mH^4}{EI_o}\,K_4 \quad , \tag{12a}$$

$$K_4 = 1 - \gamma + \gamma\,\frac{120}{11}\,\frac{1}{\alpha^2}\left[\frac{1}{3} - \frac{1 + \sinh\alpha\,(\frac{\alpha}{2} - \frac{1}{\alpha})}{\alpha^2\cosh\alpha}\right] \tag{12b}$$

This expression may be readily verified in Ref. 9.

Introducing Eqs. (10) and (11) in Eq. (9) and evaluating the numerator integral in the frequency expression, there results

$$\omega^2 = \frac{140}{11} \frac{EI_o}{mH^4} \frac{1}{K_5} \quad , \tag{13}$$

where the integral

$$K_5 = \int_o^1 z_o \psi_1 \, d\bar{\xi} \tag{14}$$

has been named in a fashion similar to Ref. 9. The evaluation of the integral Eq. (14) is laborious and not reproduced here. The final result is

$$\omega = 3.57 \sqrt{\frac{EI_o}{mH^4 K_5}} \; , \tag{15a}$$

$$K_5 = (1 - \gamma) + \gamma \frac{1}{11} \left[\frac{56}{\alpha^2} - \frac{105 \tanh \alpha}{\alpha^3} - \frac{140}{\alpha^4} + \frac{840 \tanh \alpha}{\alpha^5} - \frac{420}{\alpha^6} - \frac{420 \tanh \alpha}{\alpha^7} - \frac{840}{\alpha^5 \cosh \alpha} \left(\frac{\alpha}{2} - \frac{1}{\alpha}\right)\right] \tag{15b}$$

It may be easily seen that $K_5 \to 1 - \gamma = I_o/I_\infty$ for $\alpha \to \infty$. By writing the expression in the square bracket of Eq. (15b) as a fraction with denominator $\alpha^7 \cosh \alpha$ and by taking the 5th derivative of both the nominator and the denominator, it can be shown that the value of the expression in the square bracket of Eq. (15b) tends to 11 and consequently $K_5 \to 1$ for $\alpha \to 0$. Hence, the formula is correct in the limits.

Similar to the K_u-factor (Eq. (12b); Ref. (9)), which represents the ratio of the coupled and uncoupled top deflection, K_5 represents the square of the ratio of the coupled and uncoupled fundamental periods. It is represented in Fig. 6, together with a visualization of the shear medium parameters α and γ.

Fig. 7 shows a comparison of the approximate frequencies of Eq. (15) with the frequencies reported in Ref. (14) resulting from the numerical solution of the eigenvalue problem associated with the 6th order differential equation of motion. The agreement is excellent, and the error of the approximate solution is evidently insignificant in view of the discrepancies between theory and test results.

For two identical planar coupled walls or for a web wall coupled with flange walls, the effect of the wall shear deformations may be readily implemented in the approximate solution. In these cases the shear deformations do not influence the coupling phenomenon within the approximations of classical beam theory and result simply in an additive term in the expression for the derived deflected shape

$$z_1 = Z_1(\psi_1 + \phi_1), \tag{16}$$

where Z_1 and ψ_1 are still given by Eqs. (11b) and (11c), and $Z_1\phi_1$ denotes the shear deflections of a cantilever. From classical beam theory follows

$$\phi_1 = \frac{140}{11} \frac{EI_o}{GA'H^2} \left(\frac{3}{2}\xi - \frac{1}{2}\xi^3\right) \quad (17)$$

Introducing Eq. (16) in the denominator integral of the frequency expression results in an additional term in the expression for K_5

$$K_5 = K_5' + K_5'' \quad , \quad (18a)$$

$$K_5'' = \frac{56}{11} \frac{EI_o}{GA'H^2} \quad , \quad (18b)$$

where K_5' is still given by Eq. (15b). The interrelation between the sectional coupling degree and the shear deformation enters only in the value for the effective shear area A' and a reasonable mean value may be used. In Fig. 8 the approximate solution, Eqs. (15a) and (18), is compared with a frame analogy result reported in Ref. (20). The shear influence in the K_5-factor was 6%.

The approximate solution just presented allows an easy determination of the fundamental period of composite precast walls and coupled shear walls on a pocket calculator. The evaluation of Eq. (15b) is not too tedious: for $\alpha > 3$ $\tanh \alpha \approx 1$, and the higher order terms soon become insignificant for increasing α. The curves in Fig. 6 have been dashed for $\alpha < 1$ to indicate that at least seven digit accuracy is needed (small differences of large numbers); but this range is scarcely of practical importance.

As the square of the ratio of the coupled and uncoupled fundamental periods, the K_5-factor defines the range within which the apparent fundamental period may change when the vertical connections or the coupling beams become inelastic. While these bounds are of value, it is necessary to have a more complete understanding of the potential inelastic response of composite walls for aseismic design.

INELASTIC BEHAVIOR OF COMPOSITE PRECAST PANEL WALLS

Design Concepts

Generally speaking, the performance of a structure under severe earthquake excitation depends to a great extent on the inelastic hysteretic behavior of its weakest elements, their structural function and on the nature of the overall yield mechanism associated with their inelastic behavior. Favorable performance may be expected, if the elements exhibit stable hysteretic behavior and if the primary yield mechanism is confined by elements that are still elastic. The beneficial effects of hysteretic damping may then combine with the positive, or override the negative, effects of the longer apparent fundamental period associated with inelastic response. On the other hand, in strongly degrading elements the deterioration process is amplified by the tendency of inelastic deformations to aggregate in weak elements. If these elements are primary (i.e. gravity) load bearing elements and if the yield mechanism is not confined by other elastic elements, overall stability may be threatened. A major concern of aseismic design concepts is therefore the establishment of favorable hierachies of yield mechanisms (21).

In panelized buildings the joints are likely to be the weakest elements. It has been suggested that global slippage between wall panels along horizontal joints could be used as both an energy dissipating and force isolating mechanism (3,4). However, horizontal joints are primary (i.e. gravity) load bearing elements. Because the horizontal connections often serve to join both load bearing wall panels and the floor planks, their mechanical behavior is rather complex and also sensitive to the floor behavior. Moreover global slippage along a horizontal joint represents an unconfined yield mechanism for the configurations under investigation. Thus it seems very difficult to design horizontal joints for controlled inelastic behavior without threatening the overall stability of the wall. Earlier studies on the inelastic behavior of horizontal joints (3) indicate that global slip is only likely for very low values of shear friction coefficient and/or normal stress. It was concluded that the change in response due to inelastic action was mainly associated with the elongated apparent fundamental period and that the contribution of energy dissipation was minor. To sum up, it is felt that horizontal joints are questionable as primary energy dissipating elements and should, on the contrary, be protected from inelastic action as long as possible to ensure their primary function as load bearing elements.

With respect to vertical joints the situation is in many ways just the opposite. Vertical joints do not in general, serve a primary, gravity load bearing function. Yielding of a vertical joint is confined as long as the walls have not yet reached their ultimate strength. A poor vertical joint in one story has little effect on the overall behavior of the structure. Only the behavior of the vertical joint over the entire building height is important. The structural configurations of vertical joints are rather simple. Because only relatively little stiffness is needed to ensure nearly monolithic behavior under service level loads, mechanical connectors can be used as is usually the case in panelized systems found in both the United States and Canada. Such mechanical connectors seem particularly suited to a design for controlled inelastic behavior. The use of vertical joints as energy dissipating elements has also been suggested by Paulay (21) and Pollner (22). It is thus felt that the primary inelastic action in precast concrete panel buildings with horizontal and vertical joints should be confined to vertical joints.

For frames similar reasoning has led to the strong column, weak girder design philosophy. It is now also generally accepted that coupled shear walls are very efficient earthquake resistant structures, if the primary inelastic action is confined to the coupling beams and if the latter are properly designed to avoid premature shear degradation and failure (20,21,23). Such a coupled shear wall system may be viewed as an extreme case of a strong column, weak girder design.

In view of the above reasoning a promising design concept for panelized buidlings may be a strong horizontal joint, weak vertical joint design philosophy. If such a concept holds, the design could proceed along similar lines as for coupled shear walls. Proper selection of strength and stiffness of the vertical joints and ensuring their ability to sustain continuing inelastic excursions without significant degradation are then of primary importance.

Discussion of Vertical Joint Mechanical Properties

In this section the vertical joint characteristics, strength, stiffness and hysteretic behavior are discussed in the light of the strong horizontal joint, weak vertical joint design concept.

Strength--The selection of the strength or yield level of the vertical joint determines:

- the degree of energy dissipation while the walls are still elastic
- the degree of overall softening of the structure
- the degree of protection of the walls and horizontal joints from primary inelastic action
- the degree of protection of the joint itself from degradation

Fig. 9 shows a conceptual single-degree-of-freedom model in terms of a base shear versus top deflection relationship. If the relative coupling stiffness α is low, the connection shear distribution is relatively even (see Fig. 5a) and yielding will start more or less simultaneously throughout the connection. If α is high, the overall stiffness does not decrease significantly until the connection is yielding nearly over its entire length (23). Thus the load-deflection curve can be reasonably approximated by two straight lines in the elastic range of the walls (Fig. 9b). Line AB corresponds to the elastic response, while along line BC the vertical connection is yielding. At point C the center wall becomes inelastic, a plastic hinge forms at the base and the ultimate strength is reached. For simplicity the last part of the load-deflection curve is also represented by a straight line CG. The slopes K and pK of the load-deflection curve are proportional to the square of the coupled and uncoupled frequencies, respectively, hence $p = K_5$, where K_5 is given by Eq. (15b).

A strong horizontal, weak vertical connection design implies that the structure exhibits considerable energy dissipation before the walls reach their ultimate strength. For very high connection yield levels (e.g. line ADE) the energy dissipated in the elastic range of the walls tends to zero. Similarly, for very low yield levels (e.g. line AFG) the effect of hysteretic damping in the vertical connection becomes negligible for response amplitudes approaching the elastic limit of the walls.

Clearly, from the point of view of energy dissipation, the yield strength has to be chosen at an intermediate level (e.g. line ABC) in such a way that the maximum possible value of hysteretic damping in the vertical joint is reached slightly before the elastic limit of the walls.

To illustrate this, Fig. 9c shows the hysteresis loop of the single-degree-of-freedom model for a steady state forced vibration with amplitude $w_y < w < w_u$. For the time being, the vertical connection is assumed to have a stable, elastoplastic hysteretic behavior. Degradation is treated later. The model assumes also that the connection yield cycles are governed by the fundamental mode of the structure. At least for relatively low α-values this is confirmed by the computer studies presented later in the paper. The equivalent viscous damping of the equivalent linear elastic model with stiffness K_e (Fig. 9c) is given by

$$\zeta_e = \frac{2}{\pi} \frac{\mu-1}{\mu + \frac{p}{1-p}\mu^2} \;, \quad \mu = \frac{w}{w_y} \tag{19}$$

where ζ_e is the ratio of critical damping and the viscous and hysteretic energies dissipated in one cycle have been matched at the fundamental frequency. The equivalent damping ratio is zero for $\mu = 1$ and $\mu = \infty$ and has for $p \neq 1$ a maximum at

$$\mu(\zeta_e = \max) = 1 + (p)^{-\frac{1}{2}} \tag{20}$$

Hence, if the connection yield level is chosen such that $w = \mu(\zeta_e = \max)w_y$ is slightly smaller than w_u, the hysteretic damping expressed as equivalent viscous damping would increase with increasing amplitude of response and reach its maximum slightly before the walls become inelastic. For the structural configurations of Fig. 3 the γ-parameter (Eq. (2c)) ranges roughly from $\gamma = 1/2$ to $\gamma = 3/4$. Thus reasonable values for $p = K_5$ may range from 0.25 to 0.64 and, hence, for $\mu(\zeta_e = \max)$ from 2.25 to 3.0.

Assuming a triangular force distribution, the model yield displacement w_y represents, with reasonable accuracy ($\pm$ 12% for $\alpha = 2$ to 16), the top deflection, at which the mean connection shear flow of the linear elastic structure reaches the yield strength. Thus the connection yield level suggested by the above α values is $100/\mu$ = 35 to 45% of the mean shear flow in the linear elastic structure experiencing a top deflection $w \approx w_u$. Alternatively the yield level may be expressed in terms of the actual load level corresponding to w_u (see line EG in Fig. 9b). It corresponds to $100/(\mu K_e/K)$ = 55 to 65% of the mean shear flow in the linear elastic structure under a lateral load initiating inelastic behavior of the walls.

Clearly, the above reasoning intends only to indicate trends. The optimum value of the vertical connection yield level depends on the possibly conflicting effects of hysteretic damping and of the increased apparent fundamental period and must be established by computer studies with actual earthquake records. The last of the four points mentioned at the beginning of the section aims at mechanical connection devices as commonly used in American systems. To ensure stable hysteretic behavior such connectors should be designed to yield in the connector itself and the yield strength of an individual connector should be low enough to avoid degradation of the concrete embeddment.

Stiffness--When discussing the selection of the vertical joint stiffness, two distinct regions have to be distinguished: the low α value or sensitive range and the high α value or insensitive range. In the sensitive range the joint stiffness strongly influences the overall stiffness, the fundamental period and the amount of stiffness change pK (Fig. 9b) when the connection starts yielding.

It may often be desirable to achieve nearly monolithic behavior for service level loads. The relative joint stiffness α will then be chosen at least at the beginning of the insensitive range where the overall stiffness and even more the fundamental period are nearly insensitive to the connection stiffness. From the point of view of aseismic design such a choice has the advantage that nonstructural coupling stiffness not accounted for has no influence on the overall response (however nonstructural strength has!). An increase of α over the threshold of the insensitive range increases the connection ductility ratios and changes the connection shear distribution to a more uneven shape approaching that of beam theory (Fig. 5a). In panelized buildings there is a desire to keep details constant over the height of the buildings. For high α values it may therefore become difficult to design connections to remain nominally in the elastic range at service level on the one hand and to adopt the weak vertical connection concept on the other.

At first glance it seems also reasonable to provide the vertical joint with excessive stiffness to allow for possible stiffness degradation without entering the sensitive range. This idea is discussed in the next paragraph.

Hysteretic Behavior--

The fundamental periods of many panelized buildings are often in a range where softening of the structure means not only larger deformations but also increased seismic forces. In such situations the strong horizontal, weak vertical joint concept seems only applicable, if the connection does not prematurely fail in a brittle manner and if the relative coupling stiffness does not degrade to the sensitive range at least for excitation levels up to the elastic limit of the walls. To indicate trends, again some rough estimate is needed.

Proceeding along the lines of a paper by Paulay (11), the connection deformations can be readily estimated as soon as the connection is yielding nearly over its entire length. However they should be expressed in terms of the top deflection rather than the inertia load to relate them to the hysteretic damping characteristics as previously discussed. As shown in Fig. 10, the total connection deformation Δ may be comprehended as the superposition of the deformations due to the inertia forces p and the connection yield forces q_y acting individually on the uncoupled, but equally deflecting walls. From simple beam theory follows

$$\Delta = (w + w_o) \frac{c}{H} \phi_I' - w_o \frac{1}{\gamma} \frac{c}{H} \phi_o' \tag{21}$$

where $(w + w_o)\phi_I$ and $w_o\phi_o$ are the deflected shapes due to the inertia forces and the connection yield forces, respectively, w is the total resulting top deflection and primes denote derivatives with respect to the dimensionless coordinate ξ. The parameter γ accounting for the axial deformations of the walls is defined by Eq. (2c) and c is the distance between the centers of gravity of the walls adjoining the connection. The terms ϕ_I', ϕ_o', w_o in Eq. (21) are given by

$$\phi_I' = \frac{5}{11} (3 - 4\bar{\xi}^3 + \bar{\xi}^4) \tag{22}$$

$$\phi_o' = \frac{3}{2} (1 - \bar{\xi}^2) \tag{23}$$

$$\bar{\xi} = 1 - x/H = 1 - \xi$$

$$w_o = \frac{1}{3} \frac{q_y cH^3}{EI_o} \tag{24}$$

where a triangular intertia force distribution has been assumed. By introducing the yield displacement

$$w_y = w_o \frac{p}{1-p} \tag{25}$$

of the conceptual single-degree-of-freedom model shown in Fig. 9, the connection ductility ratio, $\mu_c = \Delta/\Delta_y$, can be related to the ratio $\mu = w/w_y$ used in the expression Eq. (19) for the equivalent viscous damping ratio.

$$\mu_c = \frac{\Delta}{\Delta_y} = \frac{w_o c}{\Delta_y H\gamma} \left[\left(\frac{w}{w_y} \frac{w_y}{w_o} + 1 \right) \gamma \phi_I' - \phi_o' \right] \tag{26}$$

But with Eqs. (2) and (24) the terms $\Delta_y = q_y/k$ and w_o are related by

$$\frac{w_o}{\Delta_y} = \frac{1}{3} \beta \frac{H}{c} = \frac{1}{3} \alpha^2 \gamma \frac{H}{c} \tag{27}$$

With Eqs. (25), (26), (27) the connection ductility ratio is finally given by

$$\mu_c = \frac{1}{3} \alpha^2 \left[\left(\mu \frac{p}{1-p} + 1 \right) \gamma \phi_I' - \phi_o' \right] \tag{28}$$

where $\quad p = K_5(\alpha, \gamma), \quad \mu = w/w_y$

This approximate expression holds as long as $\mu_c \geq 1$ over nearly the entire connection length. This is the case for $\mu \geq 1.1$ to 1.5 dependint on α and γ. Note that the relation between μ_c and μ is completely defined by the shear medium parameters α and γ. It should also be noted that all definitions and equations hold equally for two and symmetric three-wall problems, if in the latter case k, q_y and q denote the sum of the stiffness, strength and shear flow of the two connections.

Eq. (28) shows that the connection ductility ratio is strongly dependent on the relative coupling stiffness and consequently also on the connection stiffness k. The total deformation, Eq. (21), however, is naturally independent of k. For a given connection yield strength the increase of the ductility ratio with α is due to the decrease of the yield deformations Δ_y and w_y.

It is now possible to return to the original question, if degradation of the connection stiffness into the sensitive range, where the overall stiffness of the structure becomes strongly affected, can be avoided for μ-values indicating significant hysteretic damping. Fig. 11 shows a commonly used, stiffness degrading model. Also included in Fig. 11 is the definition of the ductility ratio and the cyclic ductility ratio (23).

If, for simplicity, it is assumed that $\mu_c^+ = \mu_c^- = \mu_c$, then the cyclic ductility ratio is given by

$$\bar{\mu}_c = 2\mu_c - 1 \qquad (29)$$

and for the degraded connection stiffness follows

$$k_{degr.} = \frac{k}{\bar{\mu}_c} \qquad (30)$$

The relative coupling stiffness α is proportional to the square root of the connection stiffness and, hence, for the degraded relative coupling stiffness follows with Eqs. (28), (29) and (30)

$$\alpha_{degr.} = \frac{\alpha}{\sqrt{2\mu_c - 1}} \qquad (31)$$

Fig. 11 shows the results of the evaluation of Eqs. (28) and (31) for two values of μ indicated in the discussion on connection strength, two estimated bounds to γ for panelized buildings and two values of α : $\alpha = 4$ corresponds to the threshold of the insensitive range and $\alpha = 16$ corresponds to nearly monolithic behavior.

Fig. 11 indicates a slightly improved performance for high α values. However the difference is essentially due to the fact that, for a given yield level and μ, an increase of α decreases w_y and, hence, $w = \mu w_y$. Moreover the very high ductility ratios μ_c associated with high α values seem rather theoretical. Thus providing the connection with excessive stiffness appears questionable.

Comparison of the results with Fig. 6 shows that a structure, whose hysteretic joint characteristics correspond approximately to those of Fig. 11, will always degrade into the sensitive coupling stiffness range for values of μ as low as 2. If softening means lower seismic forces, this may be an advantage. However panelized buildings are often in period ranges where softening is associated with increased seismic forces. It is questionable if the effect of hysteretic damping can compensate for the softening and caution seems appropriate with a deliberate weak joint design. On the other hand the vertical joint seems ideally suited as an energy dissipating mechanism in panelized buildings. The dilemma points to the urgent need for the development of vertical joints with stable hysteretic behavior.

Finally it must also be recognized that, when softening of the structure will lead to higher seismic forces, any noticeable coupling effect is of value. As long as a vertical connection retains any residual stiffness and strength, it can contribute to improved seismic behavior relative to a simple, noncomposite wall. Thus prevention of a premature brittle failure, such as a brittle failure of the embeddment of a connector, is of primary importance.

Computer Studies

To illustrate the trends discussed above results from several computer studies are presented.

In a first study (24) the inelastic seismic response of a U-shaped composite wall as shown in Fig. 1 and 3c was studied. Dimensions and contributing masses were derived from a ten story building similar to that shown in Fig. 1. The walls were modelled by finite elements and crack opening in the horizontal joints of the web wall was included. However it was assumed that the connection strength was insufficient to create tension in the flange wall and consequently the flange wall was assumed to remain elastic and condensed into a line stiffness. The vertical connection consisted of two mechanical connectors per story. A stiffness and strength degrading connector model (Fig. 12) was adopted. The connector characteristics were selected to roughly match reported test results (25). Viscous damping was assumed to be 5% in the fundamental mode. The shear medium theory parameters were $\gamma \simeq 0.5$, $\alpha \approx 4$, $K_5 \approx 0.6$. The fundamental periods of the composite and isolated web wall were roughly 0.4 and 0.5 seconds, respectively.

The structure was subjected to the first seven seconds of an artificial earthquake, with 1.0g peak acceleration, created to match the Newmark-Blume-Kapur response spectrum at 2% damping (Fig. 13). Note that due to the rather high frequency content of the earthquake the isolated wall experiences smaller seismic forces than the composite wall.

Fig. 14 shows the envelopes of maximum response for the following four cases: linear elastic composite wall (linear elastic); nonlinear composite wall with web wall horizontal joint opening and connector yield level at 6000 kg and 4000 kg (nonlinear 6000 and nonlinear 4000, respectively); isolated web wall with horizontal joint opening (isolated).

As expected from the response spectrum, the linear elastic composite wall sustains the largest seismic forces. For decreasing connector yield levels the combined effects of hysteretic damping and of the softening of the structure result in decreased force levels approaching those of the isolated wall. It should be noted however, that the axial couple is reduced too, thus the base moment to be sustained by the web wall itself was approximately the same for the linear elastic, nonlinear 4000 and isolated case.

The maximum deflection envelopes are particularly interesting, because the effect of hysteretic damping in the vertical connection eventually overrides the conflicting effect of softening. In the nonlinear 6000 case the hysteretic damping in the vertical connection approximately compensates the effect of softening due to connector yielding and horizontal joint opening. A further decrease of the connector yield level to 4000 kg increases the hysteretic damping enough to result in the smallest maximum deflections.

For the nonlinear 4000 case the top deflection time history is compared in Fig. 15 with the distortion time history of a connector at midheight typical for all connectors in the upper two thirds of the structure. The comparison shows that the connector yield excursions are governed by the fundamental mode.

Fig. 16 shows the hysteresis loops of the same typical connector. Note the tremendous stiffness degradation. In the last large yield excursion corresponding to the maximum top deflection, the degraded stiffness was roughly twelve times smaller than the initial stiffness. Thus the upper half to two thirds of the vertical connection would have a relative coupling stiffness α in the range of 1.2, would the walls behave elastically. It appears therefore questionable, if under continued shaking the hysteretic damping could still compensate for the loss of stiffness in the structure. The maximum top deflection of the nonlinear 4000 case corresponds to $\mu = 2.0$, where this value has again been calculated on the basis of the linear elastic wall properties. It is interesting to note that the above degraded coupling stiffness agrees well with the estimated values of Fig. 11 although the web wall behaved nonlinearly.

The second study (26) investigated the coupling of coplanar walls through floors that were assumed to be cast in place. Because the values α, γ and K_5 characterizing the structure are near or within the range of precast composite walls, the results are equally applicable to the coupling effects being discussed. Dimension and masses were again derived from a ten story building similar to that of Fig. 1. After detecting no significant differencies between a finite element and a wide column frame analogy approach, the latter was used for the inelastic studies. The walls were assumed to remain in the elastic range and inelastic behavior was confined to the equivalent coupling beams. Elastoplastic hysteretic behavior was assumed. The shear medium parameters were $\gamma \approx 0.8$, $\alpha \approx 2.6$, $K_5 \approx 0.4$. The fundamental periods of the coupled and isolated walls were 0.308 and 0.475 sec, respectively. The structure was subjected to the first six seconds of the NS component of the May 18, 1940 El Centro earthquake scaled to a peak acceleration of 0.25g. The response spectrum (Fig. 17) shows that, contrary to the previous study, the uncoupled walls have to sustain larger seismic forces than the coupled walls.

Fig. 18 presents the envelopes of maximum response for the linear elastic coupled and uncoupled walls and for four cases of inelastic coupled walls with continously reduced coupling yield level. It is significant to note that the effect of hysteretic damping overrides completely the conflicting effect of the softening structure. For decreasing coupling yield levels the maximum deflections, overturning moments and shears decrease instead of approaching the response of the uncoupled walls. Even though the maximum axial couple decreases significantly (Fig. 18c), the maximum base wall moment still decreases (Fig. 18e).

Note that for the lowest coupling yield level the maximum deflections and the maximum wall base moment start to increase again (Fig.18f and e). Hence the optimum yield level lies between 15% and 30% of the maximum elastic coupling force. Alternatively expressed, the optimum yield level lies between 27% and 51% of the mean coupling beam moment of the linear elastic response scaled to the same maximum top deflection as for the inelastic response; or it lies between 43% and 75% of the mean coupling beam moment of the linear elastic response scaled to the same base moment as for the inelastic response. These values confirm the trends indicated by the conceptual model.

The coupling beam rotational ductilities (Fig. 18g) reflect the low α value. They increase with decreasing yield level and the rate of increase grows when the deflections start to increase. Note, however, that the total coupling element deformations do not increase until the deflections increase.

The results indicate that the walls and horizontal joints of panelized buildings can efficiently be protected with very low yield levels of the vertical connection provided that the connection can be designed to exhibit stable, elastoplastic hysteretic behavior. For strongly degrading joint characteristics and period ranges where softening results in increased deformations and forces a weak vertical joint design becomes questionable.

To illustate this, preliminary results from a study now underway are presented. This study investigates the seismic response of a ten-story I-shaped composite wall (Fig. 3d). The aim is to extend the first study to other connector characteristics and earthquakes.

Dimensions and contributing masses are similar to the first study. A wide column frame analogy is used and the walls are assumed to be in the elastic range. The shear medium theory parameter are $\gamma = .68$, $\alpha = 4$, $K_5 = 0.52$. The fundamental periods of the coupled and isolated wall are 0.39 and 0.56 sec, respectively. Viscous damping is 5% in the fundamental mode. The structure was subjected to 15 seconds of the El Centro earthquake with a peak acceleration of 0.2g. Fig. 19 shows the hysteretic characteristics of two of the connector models under investigation: an elastoplastic and an extremely degrading model with very low energy dissipation.

Fig. 20 shows the maximum response values as ratios of the linear elastic response. While the response of the elastoplastic model is again significantly lower, the response values for the degrading model are always higher than for the linear elastic model. The time histories of the axial couple for both models (Fig. 21) speak for themselves. It should be noted, however, that generally coupling at even the slightest stiffness and strength levels is to be preferred to the uncoupled situation.

CONCLUSIONS

This paper has examined the role of the vertical connection in the seismic response of composite walls used in large panel precast concrete buildings. On the basis of this examination a strong horizontal joint, weak vertical joint aseismic design philosophy has been proposed.

The shear medium theory has been briefly reviewed as it applies to the coupling phenomenon in precast walls. This theory was extended to provide a simple explicit expression for the fundamental period of composite precast concrete walls and coupled shear walls. This solution is based on approximate energy methods, is suited for electronic calculator use and agrees with numerical solutions of the exact eigenvalue problem.

With the linear elastic characteristics of composite walls as a basis for discussion, the effect of connection strength, stiffness and cyclic degradation on inelastic seismic behavior were explored. This discussion, along with the proposed design philosophy, was then illustrated with the presentation of a series of computer studies. It was concluded that if the vertical connections exhibit a stable, elastoplastic behavior, the walls and the vulnerable horizontal connections can be efficiently protected by deliberately designing weak vertical joints. These preliminary conclusions clearly require further computer studies to investigate, in more detail, less favorable connection characteristics and other building configurations. In addition, the lower bound fundamental periods for the applicability of this design concept must be established, since the effect of damping decreases with decreasing periods.

While favorable behavior is to be expected for vertical connections that exhibit stable elasto-plastic characteristics this is not true for all vertical connections. For fundamental periods for which softening means larger deformations and increased seismic forces, stiffness and strength degradation and shear pinching of the hysteresis loops can result in a less favorable overall response for the inelastic system than for the linear elastic system. However, potential responses to this situation, the design of vertical connections that are strong enough to remain linear elastic or the use of non-composite (simple) walls, leads to structures whose weakest element is probably the horizontal connection. Thus the designer is forced to make a decision as to whether the primary inelastic action is to be in the vertical connection or in the more vulnerable horizontal connection endangering the structure's overall stability.

It would thus seem that there are distinct advantages to the use of a strong horizontal connection, weak vertical connection design philosophy. Such a design philosophy, as previously stated, requires vertical joints that exhibit stable hysteretic behavior that is as close to elastoplastic as possible. Unfortunately the very limited experimental data on the cyclic behavior of both wet and dry vertical connections indicate that they may have trouble meeting such criteria. It is however possible to develop vertical connectors with more favorable characteristics applicable to the suggested design philosophy. One such attempt has been the lock-joint suggested by Pollner (22). This connection while exhibiting reasonably stable behavior retains the pinched hysteretic behavior typical of shear friction mechanisms. A more promising approach is proposed by Pall, et al. (27) in which coulomb friction is developed in a limited slip bolt connection. Here the basic concept is to concentrate the inelastic action in the steel connector thus protecting concrete around the embeddment from degradation. Another approach that may provide acceptable hysteretic behavior is the use of horizontal post-tensioning across the purposely smoothed surfaces of a vertical connection.

The concept of a strong horizontal joint, weak vertical joint for the aseismic design of large-panel precast concrete structures seems quite promising. It appears possible to control the seismic response of a panelized structure in such a manner as to increase damping and limit forces induced in the precast walls. However, the further development of such a concept requires both further analytical and experimental studies. It is critical that the demands that may be placed on a 'weak' vertical connection be well understood and that vertical connections be developed that can meet these demands.

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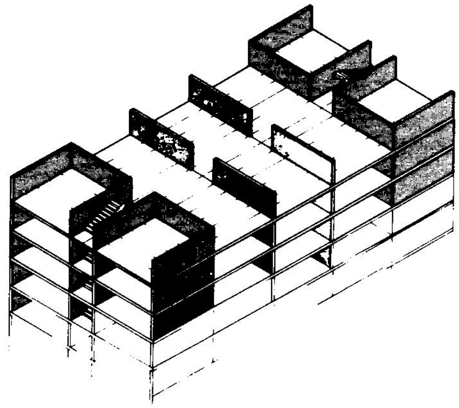


Figure 1 Large Panel Precast Concrete Building Typical of American and Canadian Construction

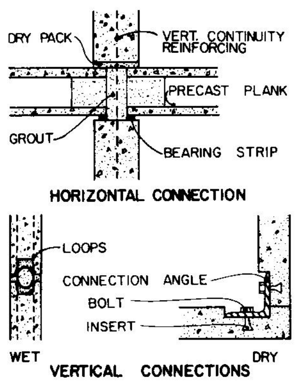


Figure 2 Typical Connections Used in Large Panel Precast Concrete Buildings

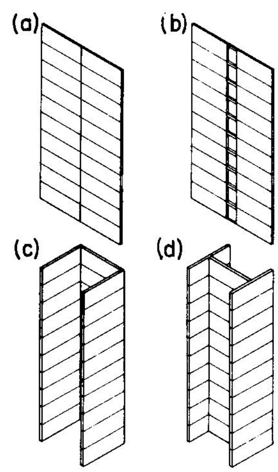


Figure 3 Typical Composite Walls

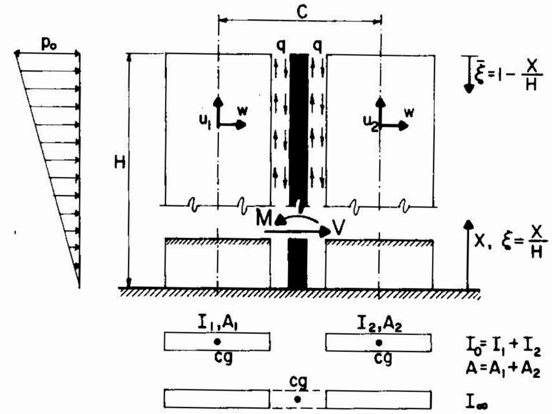
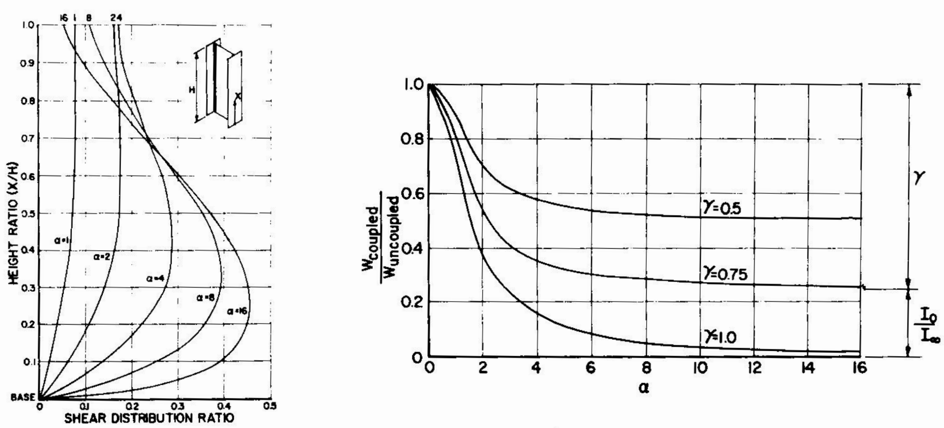


Figure 4   Shear Medium Theory - Notation



a) SHEAR DISTRIBUTION   b) RATIO OF TOP DEFLECTIONS

Figure 5   Shear Medium Theory

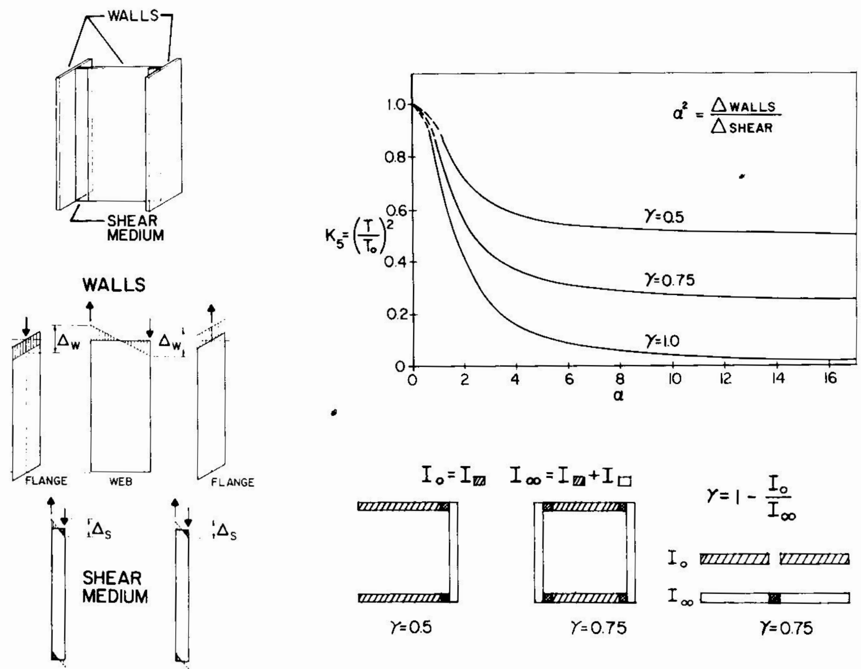


Figure 6 Fundamental Period of Composite Walls

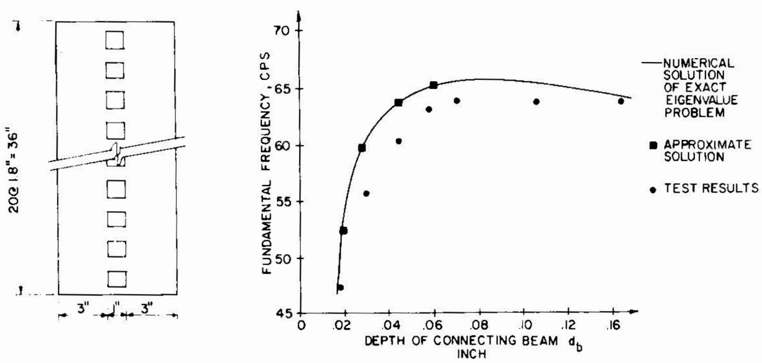
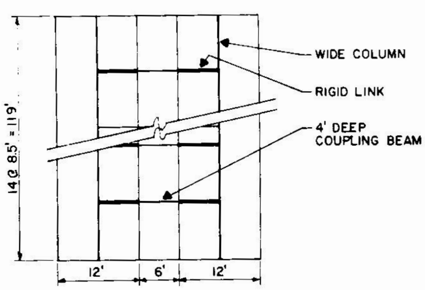


Figure 7 Comparison with Results of Tso and Chan (14)



WIDE COLUMN FRAME ANALYSIS	$\omega_1 = 21.42$
APPROXIMATE SOLUTION	$\omega_1 = 21.37$

Figure 8 Comparison with Results of Srichatrapimuk (20)

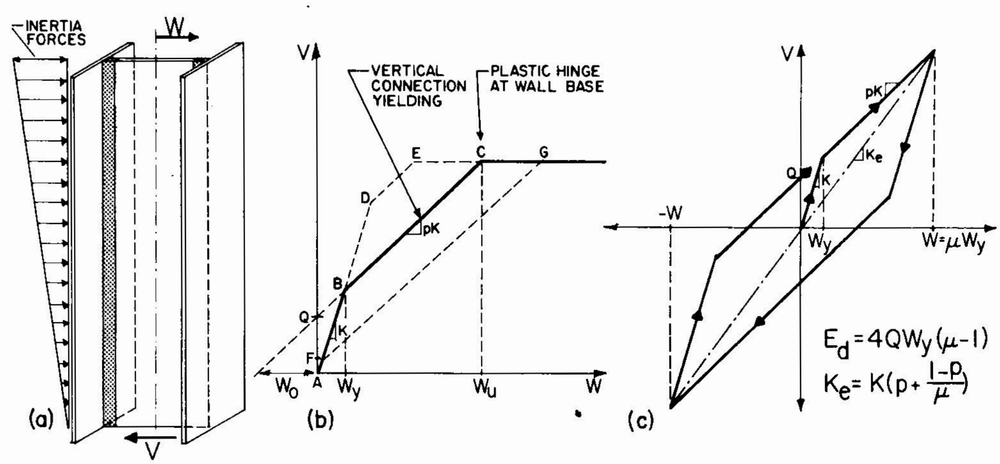


Figure 9 Conceptual Model

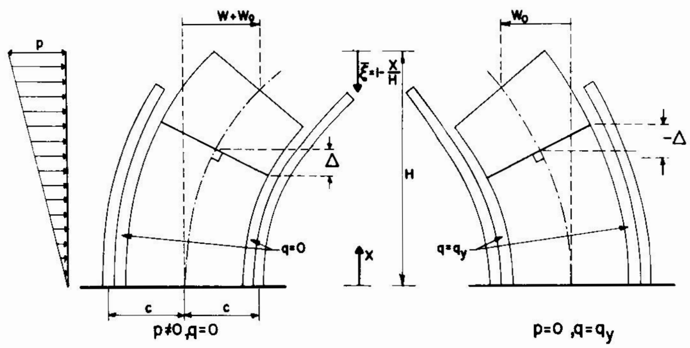


Figure 10 Connector Distortion - Notation

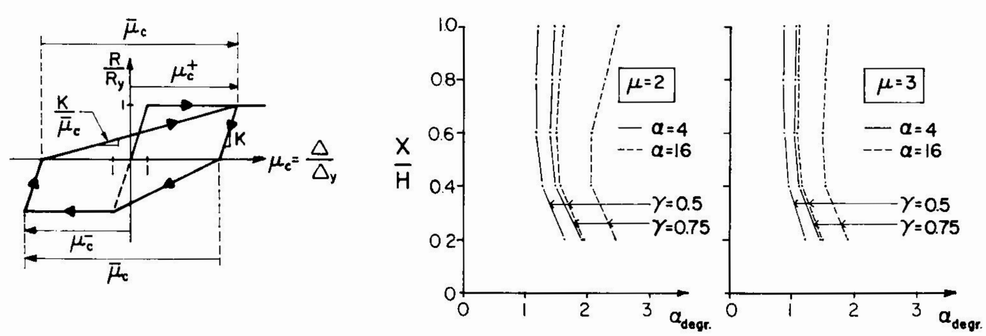


Figure 11 Effect of Stiffness Degradation

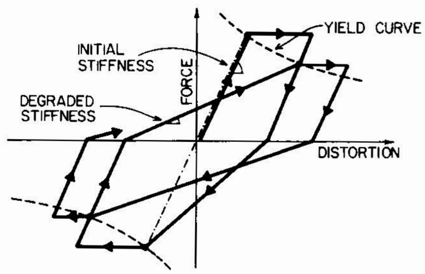


Figure 12 Degrading Connector Model

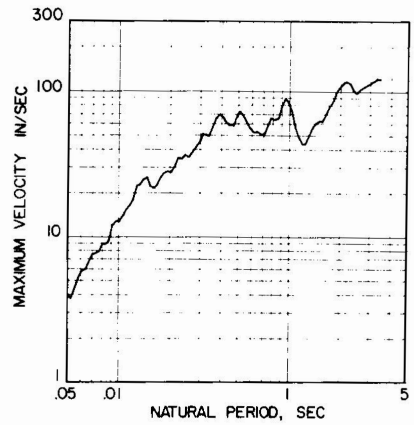


Figure 13 Response Spectrum - Artificial Earthquake, 1.0g

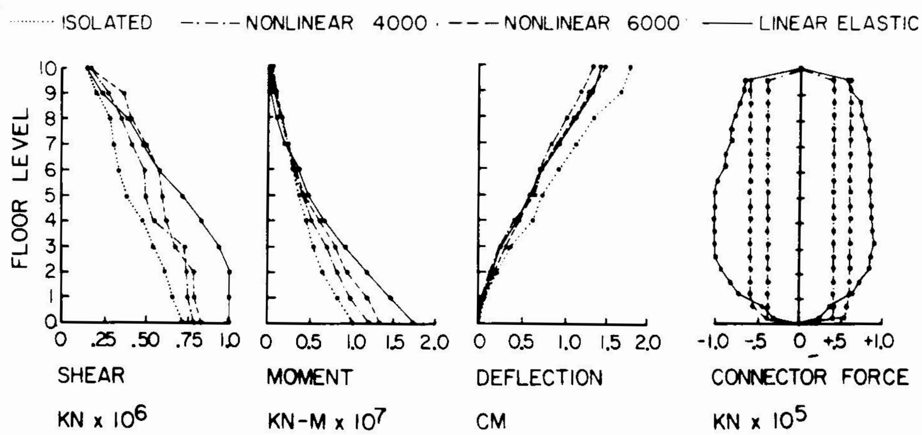


Figure 14 Results for U-Shaped Inelastic Composite Walls

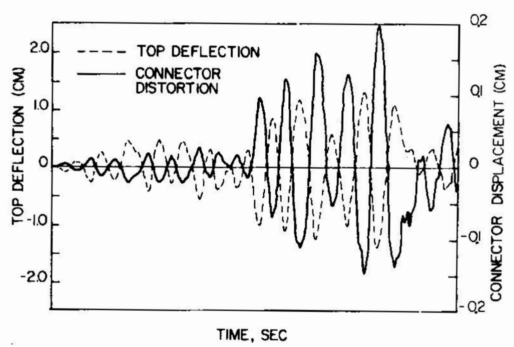


Figure 15 Time Histories for U-Shaped Inelastic Composite Walls

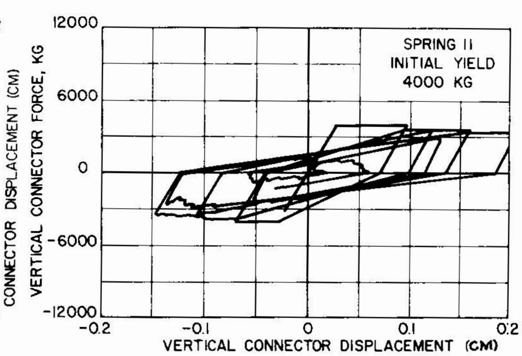


Figure 16 Hysteretic Behavior of Vertical Connection

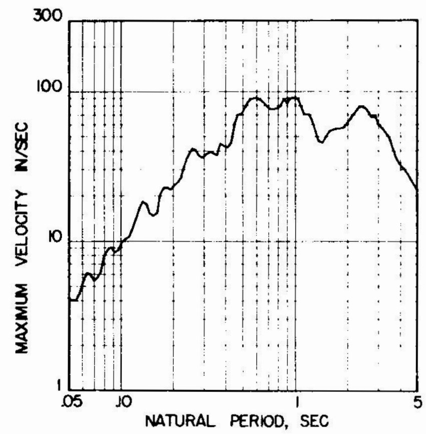


Figure 17 Response Spectrum - El Centro N-S., 0.35g

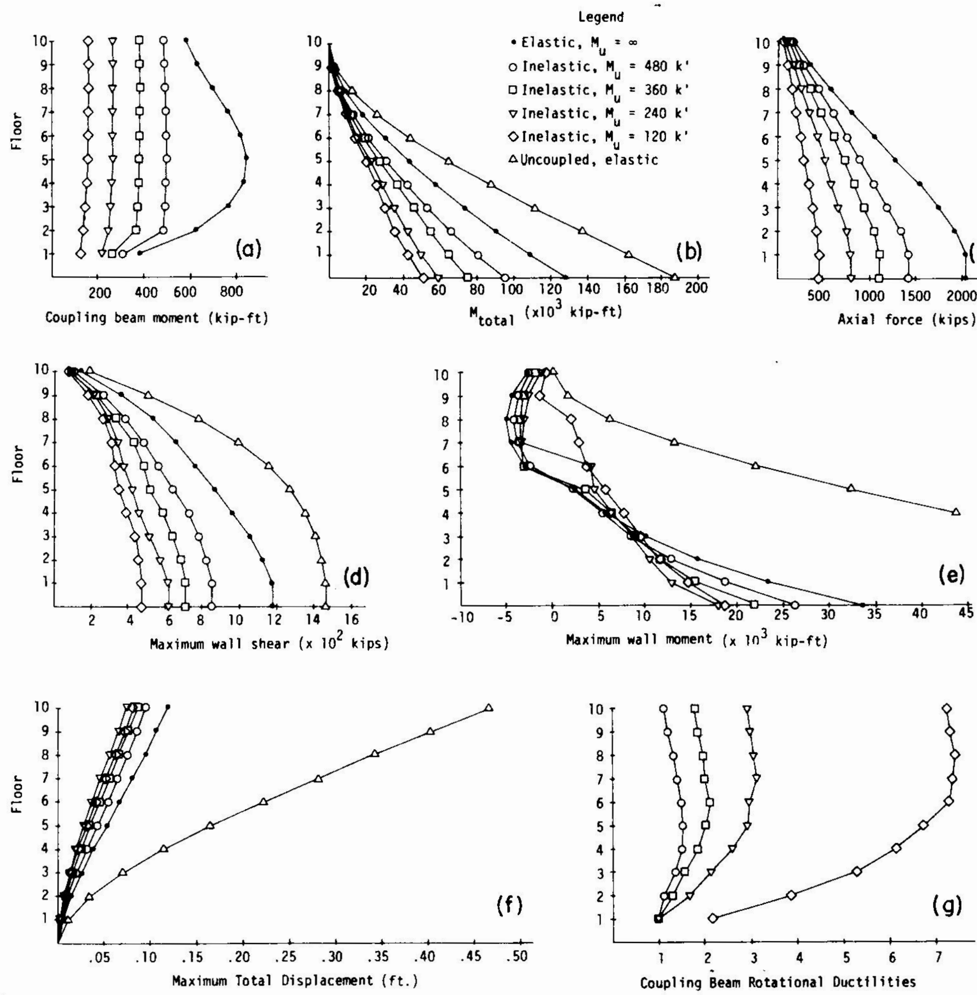


Figure 18 Effect of Inelastic Coupling Beams on Coplanar Walls (26)

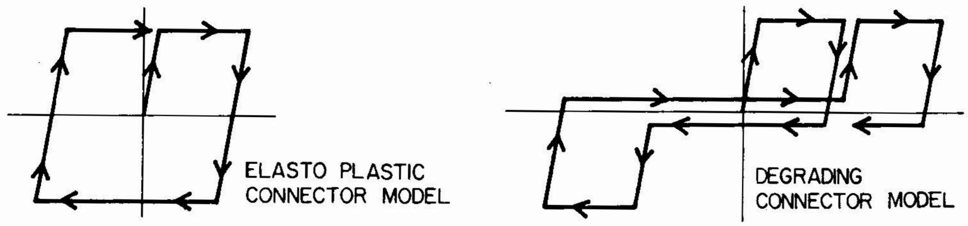


Figure 19 Connector Models

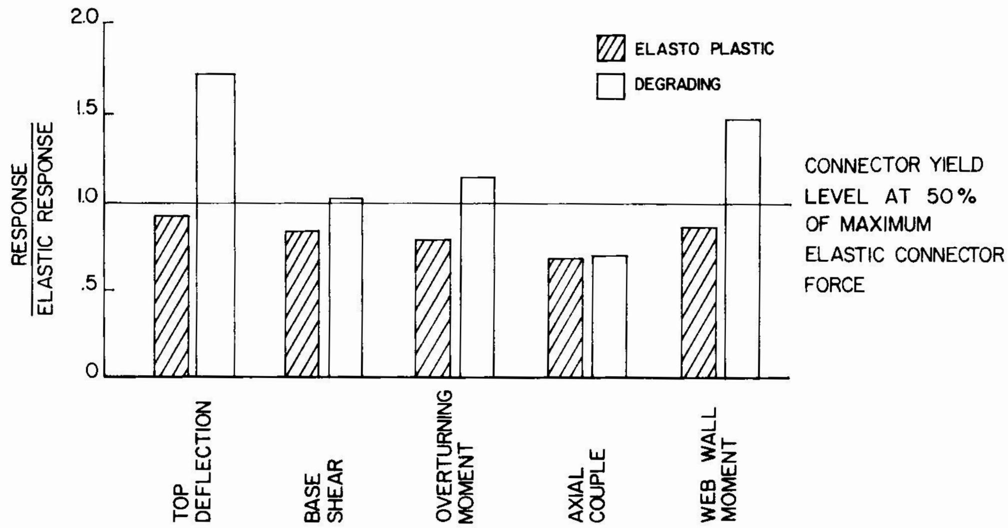


Figure 20 Results for I-Shaped Inelastic Composite Walls

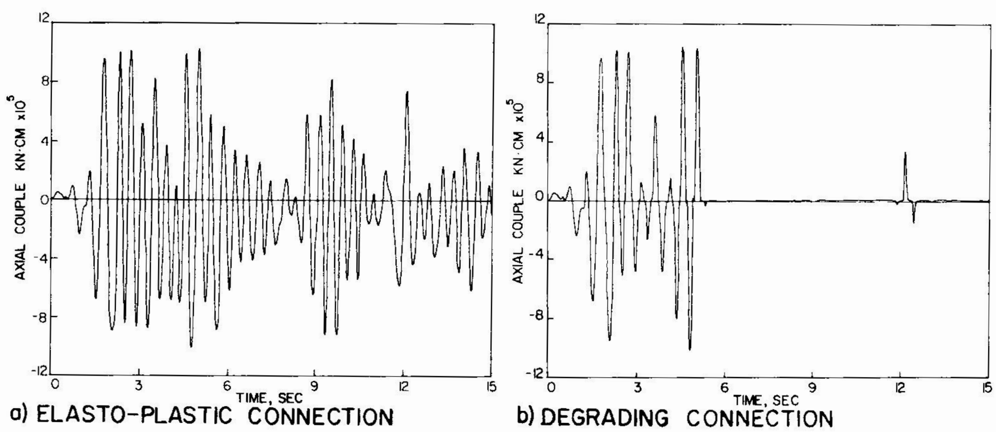


Figure 21 Comparison of Axial Couple Time Histories