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LIST OF SYMBOLS

A_s - Cross-sectional area of prestressing stand.

A_p - Contact area of panels.

C - Compression component of couple induced by cantilever moment.

C_x - Compression force at level $(x = n + 1)$.

E - Modulus of elasticity of concrete.

E_{eff} - Effective modulus of elasticity of horizontal joint.

f_{pu} - Minimum ultimate tensile strength of prestressing strand.

F_{ha} - Maximum axial force of horizontal connectors when a panel is missing.

$\overline{F}_{ha}$ - Maximum axial force of horizontal connectors when all panels are in place.

F_{hv} - Maximum shear force of horizontal connectors when a panel is missing

$\overline{F}_{hv}$ - Maximum shear force of horizontal connectors when all panels are in place.

F_r - Force in uppermost transverse tie element.

F_{va} - Maximum axial force of vertical connectors when a panel is missing.

$\overline{F}_{va}$ - Maximum axial force of vertical connectors when all panels are in place.

F_{vv} - Maximum shear force of vertical connectors when a panel is missing

$\overline{F}_{vv}$ - Maximum shear force of vertical connectors when all panels are in place.

F_x	-	Transverse tie force at level $(x, n \le x \le N)$
$F_{x\ max}$	-	Maximum transverse tie force at level $(x, n \le x \le N-1)$ for all cantilever depths.
$F_{x\ min}$	-	Minimum requirement for transverse ties.
h_s	-	Average storey height.
K_a	-	Connector axial stiffness.
K_v	-	Connector shear stiffness.
l_d	-	Width of ineffective or damaged portion of wall.
L_j	-	Thickness of horizontal joint.
M_o	-	Base overturning moment when a panel is missing (Dynamic Loading)
$\overline{M}_o$	-	Base overturning moment when all panels are in place (Dynamic Loading).
M_{ov}	-	Overturning moment induced by unsymetry.
n	-	storey number at which panel is ineffective.
N	-	number of storeys in wall.
S_a	-	Spectral value of acceleration.
T_s	-	Fundamental period of wall when a panel is missing.
$\overline{T}_s$	-	Fundamental period when all panels are in place.
T_{vf}	-	Vertical tie force due to shear friction.
T_x	-	Vertical tie force due to suspension of storey load.
T_x TOTAL	-	Total force in vertical tie.
V_h	-	Shear force on horizontal joint.
V_{hx}	-	Horizontal shear force at x, (n+1<x<N).
V_s	-	Base shear when a panel is missing (Dynamic Analysis).
$\overline{V}_s$	-	Base shear when all panels are in place (Dynamic Analysis).

V_v	-	Shear force on vertical joint.
V_{vx}	-	Vertical shear force for each panel.
W_f	-	Floor loads.
W_p	-	Weight of panel
W_r	-	Roof loads
W_s	-	Average total-storey load per meter width of wall.
x	-	Level under consideration.
α_r	-	Coefficient in figure (16).
α_x	-	Coefficient in figure (17).
β_x	-	Coefficinet in figure (19).
ϕ	-	Reduction factor, (0.9).
μ	-	Shear friction coefficient.
ν	-	Poisson's ration.
ρ	-	Mass density of concrete.
Δ_s	-	Maximum deflection of wall when a panel is missing.
$\bar{\Delta}_s$	-	Maximum deflection of wall when all panels are in place.

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Adequate low cost housing is one of the basic needs of any society and industrialized building systems appear to be a possible solution to this problem. A very successful form of industrialized building systems, known as panelized building was developed in Europe in response to major housing shortages at the end of World War II. Panelized buildings are constructed out of large precast concrete panels which are used in a vertical position as shear-wall elements and in a horizontal position as floor or roof elements. Originally, panelized buildings were designed for non-seismic regions but their potential as a solution to major housing problems prompted their use in seismic regions of the world.

The major concern in panelized building systems constructed in seismic regions is the performance of the stabilizing shear-walls built out of panels and the connecting components at the interface of these panels. The normal practice is to design the structure to accommodate static or quasi-static equivalent loads according to relevant codes, but when abnormal loadings, not specified in codes, occur, the result is usually catastrophic.

Catastrophic failure is the result of a local failure progressing horizontally or vertically and creating consequences much greater than the initial failure. This phenomenon is known as progressive collapse. In the past, research was concentrated on progressive collapse due to static loadings but recent studies are being directed towards the behaviour of panelized structures under seismic loadings.

Literature on panelized structures ranges from the general requirements for construction and planning to the detailed investigation of joints and connections under seismic loadings. Lewicki [1] outlined the principal considerations for planning and design of panelized structures, particularly buildings with large panels. The Portland Cement Association [2] presented a series of four reports on the "Methodology for designing of panelized structures". Becker et al.[3] explored the basic nature of the response of both simple and composite precast shear-walls and reported observations on the fundamental behaviour of precast walls and the implications for seismic design. Frank [4] investigated the dynamic response of large panel structural systems. The study examined the dynamic response for parameters such as height of building, width of panel, relative stiffness of the connections, the percent critical damping and variation in the expected ground motion. In the same research program, Zeck [5] investigated the joints in large panel structures in relation to the forces they are required to transfer under various loading conditions. Pall [6] developed limited-slip bolted joints for large panel structures and investigated the seismic response of buildings using this type of joint. Huttelmaier [7] studied the dynamic response of a large panel structure as influenced by varying connection stiffness and panel types.

1.2 SCOPE OF THIS STUDY

This report presents the results of a study of a twelve storey panelized shear-wall with respect to progressive collapse. The analysis is done by a method based on simple cantilever assumption, as developed in reference (2), for static loading and by a finite element analysis for both static and dynamic loadings. The aim of the study is to determine forces developed in tie and connector elements arranged to prevent progressive collapse when a load bearing element becomes ineffective and also to determine the connector forces due to a dynamic loading with panels ineffective at various levels.

First, panelized building systems and components are defined in chapter 2 and mechanisms to prevent progressive collapse and methods for calculating forces in tie elements are outlined in chapter 3. Chapter 4 presents the properties of the wall to be analysed and chapter 5 presents the results of the investigation.

For the analysis, the wall system is assumed to be rigid at the base and to behave as a cantilever with structural integrity achieved by horizontal and vertical connectors at each floor level. The panelized wall is twelve storeys high and three bays wide. Each panel is one bay by one storey in size (figure 1) and the behaviour of the structure is limited to the elastic range. For the static analysis, gravity loads consist of superimposed floor loads and weight of floors and wall panels. For the dynamic analysis the wall is subjected to a ground acceleration represented by a response spectrum.

CHAPTER 2

PROPERTIES OF PANELIZED BUILDING SYSTEMS

2.1 GENERAL

Precast elements in concrete are members cast in a factory or on site but not in their final position. Precast concrete elements exist in many forms from concrete masonry blocks to flexural and compression members, to entire rooms and buildings. Within this range, the components known as large panels exist.

Large panels are flat concrete members which perform structurally in a horizontal position as floors or roofs and in a vertical position as walls at least one storey in height. Lewicki [1] describes three levels of large panel concrete construction. They are: prefabricated floors and roofs assembled with traditional bearing walls; precast floor, roof and wall panels with finishes on site; and precast floor, roof and wall panels with finishes done at the time of casting.

The last two levels are panelized structures. A panelized structure is therefore a system with walls assembled out of panels, and roof or floors assembled out of panels or planks and with walls transferring the loads to the substructure. Unlike traditional shear-walls which commonly resist only lateral loads, large panel walls are also load bearing walls.

2.2 PANELS

Panels in industrialized buildings are basically flat plates. When used as floor or roof members in the horizontal position, loads are resisted by out-of-plane behaviour. They are flexural members and can be designed to resist vertical loads through unidirectional or bidirectional action. When used in the vertical position in shear-wall structures large panels ensure strength and stability of the building through in-plane stiffness. Out-of-plane resistance is neglected.

The overall dimensions of large panels are governed by practical considerations such as transportation, factory and assembly capabilities or architecture. In width, panels can vary in dimension from one bay to the entire width of the wall. In height, the panels in a wall have to be at least one storey. Thickness can vary according to the position of the panel within the height of the building and is proportioned according to structural requirements.

Large panels for structural walls are usually homogeneous and made of ordinary concrete. Over the years reinforcement in large structural panels has reduced considerably because research and experience [1] proved that panels for walls can be designed as plain concrete walls similar to block walls. However, reinforcement is necessary to accommodate other conditions such as erection and transportation stresses and localized effects such as at connectors and regions of stress concentration. Panels can be ribbed to increase stiffness in critical regions such as at extremeties of the wall.

2.3 JOINTS

2.3.1 GENERAL

In order for joints to participate in the integrity of a panelized structure, they are expected to withstand the same forces that would exist in the corresponding sections of a monolithic system. Since the wall system in a shear-wall building assures the physical stability of the entire structure, jointing between wall elements is significantly more important than jointing between floor or roof elements. In prefabricated buildings, joints along the interface between the wall panels and the floor slabs are called horizontal joints and the vertical interfaces between wall panels are called vertical joints.

The design of jointing in large-panel structures draws most attention because they contribute to the behaviour of the system much differently from the commonly understood monolithic shear-wall. The important features of joints are the ability to transfer loads from one element to another and the amount of deformation they permit. Particular concern is focused on their performance under severe dynamic loadings and their resistance to progressive collapse. To prevent progressive collapse, joints must be capable of bridging the gap created by failure of other members and withstanding the impact created by debris caused by some local failure.

Joints may be constructed from plain concrete, reinforced concrete or steel depending on the nature of stresses to be transferred and the practicality of installation. Plain concrete joints are essentially capable of transferring normal compression or shear stresses while reinforced concrete or steel joints are also capable of transferring normal tensile stresses.

2.3.2 HORIZONTAL JOINTS

Horizontal joints in prefabricated structural walls act as transfer mechanisms for loads from upper storeys and floor slabs to lower panels and to the substructure. Figure (2) shows some configurations of horizontal wall joints. A recommended design approach is to spread the load from the upper levels to the lower panels over the entire area of the joint (figures 2a,b,c,d,& e). When loads are transferred through two or more levels (figure 2f) or when loads are transferred on one plane but through concrete of different strengths (figure 2g), the load may concentrate at only one level or on the concrete of higher strength. Figure (3) shows a practical detailing procedure ensuring axial load transfer to the load bearing panel as well as facilitating proper grout filling. A special horizontal joint occurs between floor slabs parallel to their span (figure 2h).

Horizontal joints where floor slabs are supported on walls are subjected to additional local stresses due to the deformation of the slab and consequently rotation at the supports. Because of the rotation the contact area of the loaded surface is reduced. In such cases, Lewicki [1] estimates that the carrying capacity of the wall may be reduced by 1/3 or 1/4 of the carrying capacity when the contact surface is uniformly loaded. Alternately, when floor slabs are continuous over the support wall or when moment is transferred to the joint, Zeck [5] demonstrated the consequences on the lower panel (figure 4). Under these conditions the joint will be subjected to tension-compression stresses due to the moment connection as well as compression from gravity loads. The result could lead to tensile splitting stresses in the lower panel and should be accounted for by reinforcement.

2.3.3 VERTICAL JOINTS

Vertical joints extend to the full height of the panel, normally from floor to floor. The principal function of vertical joints is to resist vertical differential displacements of the panels. In order to resist vertical movement between panels, the vertical joints transfer vertical tangential forces [1]. Vertical movement between panels occur as a result of in-plane bending due to lateral loading (figures 5a & b), because of unequal gravity loads on adjacent panels (figures 5c & d) or due to differential settlement of the foundation. Vertical joints are also expected to resist tensile forces due to the buckling tendencies of adjacent walls (figures 5e & f).

The variation in stiffness of vertical joints affects the entire wall structure considerably. Very stiff joints restrain any differential movements permitting the wall to behave as a monolithic system (figure 6a). On the other hand, very soft joints permit movement between the panels resulting in the stiffness of the shear wall being the sum of the stiffnesses of the individual panels (figure 6b), [5].

Vertical joints may be classified as either wet or dry. Wet joints require in situ concrete or gout to fill the gap between adjacent panels and can be reinforced or unreinforced. With respect to progressive collapse, unreinforced wet joints offer no resistance and therefore are not recommended. Figure (7) shows four examples of wet reinforced joints.

In dry joints, the connection is made by welding or bolting steel sections to inserts cast in the panels. Figure (8) shows examples of welded and bolted dry joints.

2.4 GENERAL BEHAVIOUR OF LARGE-PANEL STRUCTURES

In buildings constructed of prefabricated panels, stability against lateral vibrational loadings (wind and seismic) is established by interaction between the floors and the shear-walls. The floors are usually considered stiff in their own planes, distributing lateral loads of the building to the walls.

In planning the structural system of panelized building, importance should be given to the fact that the distribution of loads between the shear-walls depends on the relative stiffnesses and the position of the walls. Also of equal importance is the fact that the load capacity of the total system depends on the capacity of the individual panels as well as the interaction between them.

For buildings of monolithically cast shear-walls, these problems are normally easily handled. However, for buildings erected from panels, the problems are more complicated because the strength and stiffness of the joints are quite different from those of the panels. The stress distribution in the wall will depend upon the shear stiffness of the joints and the strength of the wall will be affected by the presence of these joints.

Panelized highrise buildings therefore need special considerations in assuring the structural integrity of the entire structure because the monolithic shear-wall is now replaced by discrete elements. Further problems arise when it is necessary to remove one or more panels for architectural or functional purposes or when one panel becomes ineffective. This situation is analogous to having openings in a continuous shear-wall. In such cases connection systems should be designed for progressive collapse conditions in order to redistribute loadings to alternate support elements.

2.5 IMPORTANCE OF WALL ARRANGEMENT

To ensure the integrity of a panelized structure, the individual components of the structure should behave as a continuous unit. Since structural stability is ensured by the wall elements, wall assemblies should be so arranged that they are in two vertical perpendicular planes.

At least two such wall should exist in one plane in order to avoid complete failure of the entire building if one wall fails. Whenever possible the system should be symmetric to avoid torsional effects [2]. The arrangement of support wall systems can be classified into two main categories and many variations of these can be developed for a particular situation. They are the main walls in the long sense of the building (the longitudinal wall system), (figure 9) or the cross-wall system with the main walls in the transverse sense of the building, (figure 10).

2.6 LOADINGS ASSOCIATED WITH LARGE-PANEL STRUCTURES

The category of loadings associated with large-panel structures differs from conventional construction because of the nature of systems approach to industrialized buildings. The phases of construction take place in two different places, that is, preparation of elements in a factory and erection of components on site. Lewicki [1] describes three stages of work which determine the different loading conditions.

Stage A - De-moulding and transportation of components;

Stage B - Erection;

Stage C - Function of the individual components and service condition of the building as a whole.

Stage C is the predominant condition which influences the performance and method of construction of the individual components as well as the building as a whole and therefore forms the basis of design calculations. Stages A & B are secondary effects with respect to the structural behaviour but can be significant to individual elements when support conditions differ from the final structure.

CHAPTER 3

PROGRESSIVE COLLAPSE OF LARGE PANEL WALLS

3.1 CAUSES OF PROGRESSIVE COLLAPSE

Progressive collapse is the condition when an initial local failure spreads to other areas of the structure eventually causing structural collapse disproportionate to the initial cause or the initial damage. Loads usually associated with progressive collapse are those that are not included in codes and therefore may not be considered in design. Such loads are due to abnormal events such as gas explosions, boiler failures or ignition of some industrial liquids [8], vehicular impact, falling or swinging objects (usually during construction or demolition), adjacent excavation or flooding that cause severe local foundation failure, very high winds, sonic booms and design or construction errors. In addition, the effect of fires, earthquakes and corrosion, which are specified in codes, may also cause progressive collapse.

To preclude catastrophic failure the designer must first distinguish the difference between general and progressive collapse with regard to the initial effect and then consider rational means of limiting the spread of the local effect. The common means of preventing progressive collapse, especially in panelized structures, is to provide continuous ductile load resisting elements and connections with inherent ductility and strength and to provide significant alternative paths for load transfer by bridging the failed element [9]. The mechanisms that are commonly employed to resist progressive collapse are discussed in the following section.

3.2 MECHANISMS TO PREVENT PROGRESSIVE COLLAPSE

The major causes of progressive collapse as outlined in the previous section indicate the difficulty in setting limits with respect to this phenomenon. However, the following limitations are suggested in "Commentaries on part 4 of the National Building Code of Canada 1977"[9].

a) Where progression might be vertical, failure should be limited to the storey immediately above and below the location of the abnormal event.

b) Where the progression might be horizontal, failure should be limited.

i) to the truss, beam or precast floor or roof panel initially damaged and perhaps to one on either side, or

ii) to one bay-sized floor or roof slab or where the principal support at one end of the slab is removed, to two bay-sized slabs which may hang together as a centenary.

The following four general considerations to be used in design to avoid progressive collapse are also suggested.

1) Reduce the probability of occurrence of an abnormal event,
2) Design using ductile connections,
3) Design to resist abnormal loads, and
4) Design for alternative paths.

The fourth suggestion is fast becoming the accepted normal practice because it is easier to apply and control. This alternative path is achieved by having a proper wall arrangement as outlined in section (2.5) and by providing continuous ductile ties; around and near the periphery of floors and roofs and around the periphery of significant openings; in the longitudinal and transverse sense of slabs and in the vertical direction in structural walls. The forces in these ties are obtained by predicting typical damage caused by abnormal events and finding alternative paths around damaged areas. Based on practical design approach, ties are treated separately for walls and floor systems even though these two components behave interdependently to guarantee the overall stability of large panel structures. Details and performance of ties are discussed in the following sections.

3.3 TIES IN WALL SYSTEMS

3.3.1 GENERAL

Large panel wall systems are susceptible to progressive collapse because of the nature of their composition, i.e. discrete elements [10]. Therefore, continuity between elements is not only necessary to provide an integral structure but is also required to be able to redistribute support conditions from a failed support member to other sound elements without the consequence of overburden. When a load bearing interior panel of a multipanel wall system fails, the panel above is required to redistribute the loadings from above by beam or arch action [2 and 9], (figure 11). In this case, tensile stresses may develop and resistance is achieved by tensile elements along the horizontal joints between panels. A more critical situation arises when an exterior panel fails thereby requiring the system above to behave as a cantilever (figure 12). To create the cantilever effect significant tensile stresses are developed and this is resisted only by tensile elements which run along the length of the horizontal joint transverse to the floor span, called transverse ties [2].

In order for the system to perform as an effective cantilever, sufficient horizontal shear resistance of the horizontal joints must be developed. In the vertical direction, tensile elements called vertical ties are installed to create a clamping effect of the panels to increase the shear friction of the joint. Ties in wall systems therefore mainly resist tensile stresses created by the cantilever action (transverse ties) and act as dowels to ensure full development of horizontal shear resistance (vertical ties) [2].

3.3.2 TRANSVERSE TIES

Transverse ties are of significant importance to creating alternative paths in wall systems because they provide 100% of the tensile resistance required in the cantilever mechanism when an exterior panel fails. The installation of transverse ties along the horizontal joint is facilitated during assembly of panels and before grouting. Figure (13) shows a practical arrangement for transverse ties. Since these elements are required to resist high tensile stresses, unstressed prestressing strands are recommended because high resistance can be achieved by small diameters [2].

3.3.3 VERTICAL TIES

The principal functions of vertical ties are to ensure shear friction action between panels at horizontal joints by providing a clamping action between consecutive lifts of panels and to resist tensile forces created by overturning moments of the entire wall system [2]. Vertical ties run through the height of panels and are joined at horizontal joints (figure 14). Large diameter bars which reduce the number of elements and connecting points facilitate assembly and are therefore preferred.

3.4 REACTIONS CREATED BY CANTILEVER ACTION

For an extreme case where a full exterior panel becomes ineffective (figure 15a), a cantilever system is created [2]. For a static loading case the cantilever is required to resist loads from the roof (W_r), loads from the floors (W_f) plus weight of panels (W_p) (figure 15c). The consequence of this is that a moment and shear force are induced at the vertical joint adjacent to the failed member. The shear force created at the vertical joint (V_v) is equal to all loads carried by the cantilever and the moment at the vertical joint creates a couple consisting of a tensile force (F_r) at the top of the cantilever, tensile force (F_x) at intermediate floor levels, and a compression force (C) at the bottom, (figure 15c). Because of the failed member, the system has now become unsymmetric and an overturning moment (M_{ov}) is induced.

To ensure cantilever behaviour the tensile forces F_r and F_x, ($n \leq x \leq N-1$) induced by the moment are resisted by transverse ties. The vertical ties assist by providing adequate shear capacity (V_h) between successive panel heights and by resisting the overturning moment M_{ov}. The shear at the vertical joint V_v is resisted by the shear capacity of the vertical joint and the compressive force (C) is transferred through the vertical joint by an effective compressive block, (figure 20).

The effectiveness of the vertical ties ensures that the cantilever is rigid and that the opening of the vertical joint is linear which means that the difference in strains in the transverse ties from floor to floor is also linear with a maximum at the top of the cantilever and zero at the bottom, (figure 15b).

Formulae for calculating the reactions induced by the cantilever action, as indicated on figure (15c) were developed in reference [2], based on the following assumptions.

1. The cantilever portion of the wall behaves in an essentially rigid manner such that:
   - rotation occurs about the base of the cantilever;
   - shear slip in the horizontal joints is not significant;
   - opening of the vertical joint is linearly related to the distance from the base of the cantilever.
2. The entire wall assembly (cantilever and support) is laterally supported at each floor level by the remainder of the structure.
3. Floor-to-floor heights are approximately equal.

A linear elastic response is assumed and the compression reaction is idealized as a single force at the root of the cantilever. The equations (developed in ref.2) for calculating these reactions are presented and discussed in the following sub-sections.

3.4.1 FORCES IN TRANVERSE TIES

Forces in transverse ties are given by equations (3.1), (3.2a) and (3.2b). These equations were developed based on the equilibrium of forces shown in figure (15c).

$$F_r = \alpha_r \frac{W_s l^2 d}{2 h_s} \qquad (3.1)$$

$$F_{x(max)} = \alpha_x \frac{W_s l^2 d}{2 h_s} \qquad (3.2a)$$

$$F_x = F_r \frac{(x-n)}{(N-n)} \qquad (3.2b)$$

where,

W_s = Average total-storey load per meter width of wall (Wall load plus floor dead and live loads)

l_d = Width of ineffective or damaged portion of wall i.e. unsupported length of wall panel.

h_s = Average storey height.

F_r = Tensile force in uppermost transverse tie.

F_x = Tensile force in transverse tie, (for $n \leq x \leq N - 1$)

N = Number of storeys in wall

n = Storey number where panel is ineffective

α_r = Coefficient given in figure (16).

α_x = Coefficient given in figure (17).

x = Floor level under consideration. (See fig.1 for floor levels)

Equation (3.1) gives the tensile force in the uppermost transverse tie for varying depths of cantilevers, equation (3.2a) gives the maximum tensile force in the transverse tie element at any level $(x, n \leq x \leq N)$, (for all cantilever depths), and equation (3.2b) gives the force in the transverse tie at any level $(x, n \leq x \leq N-1)$ for a particular cantilever depth (N-n). Figure (16) gives α_r values for equation (3.1) and figure (17) gives α_x values for equation (3.2a). Examination of figure (16) indicates that the force in the uppermost tie element is maximum for the minimum depth of cantilever and decreases with increasing depth.

Based on investigation of various heights of wall, α_x was found to be small ($\alpha_x < 0.1$) for floor levels below (N-7). As a result the following minimum values are recommended to ensure overall integrity at each floor level.

a) When using equation (3.2a) to calculate the maximum transverse force, α_x should not be taken less than 0.1; and

b) in the design of the transverse tie, the force F_x should not be taken less than 80 kN (18 kips).

The second value is independent of the configuration of the assembly and is based on a 9.53 mm (3/8") diameter prestressing strand which is chosen as an arbitrary minimum size. $F_{x_{min}}$ is found from equation (3.2c).

$$F_{x_{min}} = \phi \ (A_s) \ f_{pu} \tag{3.2c}$$

where,

$F_{x_{min}}$ = Minimum requirement for transverse ties

ϕ = reduction factor = 0.9

A_s = Cross sectional area of prestressing strand=51.6mm²

f_{pu} = Specified minimum ultimate tensile strength of prestressing strand = 172 kN/mm²

Figure (18) shows a plan view of transverse tie arrangements.

3.4.2 FORCES IN VERTICAL TIES

Horizontal shear capacity required to ensure the cantilever behaviour is assured by the dowel action of the vertical ties. The horizontal shear force developed is given by equation (3.3).

$$V_{hx} = \beta_x \frac{W_s \ l^2_d}{2 \ h_s} \tag{3.3}$$

V_{hx} = Horizontal shear force at x, $(n + 1 \leq x \leq N)$

β_x = Coefficient given in figure (19).

Equation (3.3) gives the horizontal shear force for any floor levels x, $(n + 1 \leq x \leq N)$ above the ineffective panel and figure (19) indicates that the maximum value for β_x increases for levels further away from the top of the cantilever. Due to shear friction action, the horizontal force induces a tensile force in the vertical tie element given by:

$$T_{vf} = \frac{V_{hx}}{\mu} \tag{3.4}$$

T_{vf} = vertical tie force due to shear friction

μ = Shear friction coefficient specified in ref. [11]

= 1.4 for concrete cast monolithically,

1.0 for concrete placed against hardened concrete and

0.7 for concrete placed against structural steel.

In addition to the tensile force due to shear friction, vertical ties are subjected to a direct tensile force given in equation (3.5). T_x is the force due to suspension of each storey load.

$$T_x = W_s \, l_d \tag{3.5}$$

The total tensile force in vertical ties (equation 3.6) is therefore the sum of the tension due to shear friction and the tension due to suspension of a storey.

$$T_{x\,(TOTAL)} = T_{vf} + T_s \tag{3.6}$$

3.4.3. FORCE IN THE EFFECTIVE COMPRESSION BLOCK

The compression component of the couple induced by the cantilever moment is transferred to the adjacent vertical panel stack by an effective compression block at the bottom of the cantilever. Since any panel within the height of the wall may become ineffective, this compression block may occur at the base of any exterior panel. The force is given by equation (3.7).

$$C_x = \beta_x \frac{W_s \, l_d^2}{2 \, h_s} \tag{3.7}$$

C_x = Compression force at the base of any cantilever height, $(x = n+1)$

The force C_x is equal to the shear force at the lowest level in the cantilever and to ensure that this force is effectively transferred to the adjacent stack of panels, dry packing of the bottom quarter of each panel is recommended, (figure 20).

3.5 VERTICAL REACTIONS

At the vertical joint, a vertical reaction equal to the cantilever weight plus floor loads and given by equation (3.8), is induced [2].

$$V_{vx} = W_s l_d \tag{3.8}$$

V_{vx} = Vertical shear force between levels x and x + 1, $(n \leq x \leq N - 1)$.

Examination of equation (3.8) indicates that V_{vx} is constant for constant load per floor and uniform panel sizes.

3.6 TIES IN FLOOR AND ROOF SYSTEMS

Floors and roof systems, as stated earlier, provide lateral support to wall systems thereby assuring the overall stability of the building. With respect to progressive collapse, floor and roof systems do not contribute directly to bridging ineffective areas but adequate resistance is required to prevent further abnormal loadings of debris [2]. To prevent complete failure of slabs, tensile elements are required to ensure continuity between the slab components and the remaining stable part of the structure even though allowing considerable slab damage and deformation. When support elements fail, continuity of slabs is provided by ties placed between slab joints in the direction of span and these are called longitudinal ties. Floor systems also require peripheral ties which are more of a practical rather than a structural element since they act as a continuous tensile ring to hold the slab elements together as an effective diaphragm. Tensile forces in the peripheral ties were found to be negligible and only practical requirements such as installation are imposed. At extremities of the building layout, transverse and longitudinal ties can be used as peripheral ties. Since this report is limited to shear-wall analysis details of floor and roof tie elements are not presented in detail.

CHAPTER 4

PROPERTIES OF THE TWELVE STOREY SHEAR WALL INVESTIGATED

4.1 GENERAL

Based on the discussions of the previous chapters, the rest of this report deals with the investigation of a twelve storey panelized shear-wall with respect to progressive collapse and gravity and seismic loadings. The analysis is performed using a finite element computer program (described in section 4.5) for both seismic and static loadings as well as by employing the simple cantilever for static loadings described in chapter (3). The aim of the investigation is to understand the structural response of a panelized shear-wall by comparing results from the method in chapter (3) and the finite element analysis under static and dynamic loadings for panels missing in a manner to cause progressive collapse.

The shear-wall selected for investigation is a twelve storey panelized wall with elevation shown in figure (1). The program of study is shown in table (1) and all analyses are based on linear elastic behaviour.

The entire wall structure is constructed from precast concrete panels and joint properties in the form of axial and shear stiffnesses are lumped at 10 nodes per panel (figure 21). Each panel is therefore defined by three connector elements along the horizontal joints and two along the vertical joints. The wall is composed of 36 panels, three in the width and 12 in the height, and is 11 meters wide and 35.64 meters high. For the seismic analysis, a ground motion represented by a response spectrum (described in section 4.4), is imposed on the structure and for the static analysis gravity loads including the weight of the wall and dead and live loads from the floors are used, (see details of tables 2&3).

4.2 GEOMETRIC AND MATERIAL PROPERTIES OF PANELS

Each panel is 3.67 meters wide, 2.97 meters high and 0.2 meters thick. The panels are constructed from normal weight concrete of mass density $\rho = 2.4 \times 10^3$ kg/m^3, Poisson's ratio $\nu = 0.17$ and modulus of elasticity $E = 27.6 \times 10^6$ kN/m^2. Floor and roof slabs are assumed to be of same thickness and material properties. At each floor level, the slabs are assumed to transfer a uniform load to the wall panels equal to 64.11 kN per meter width of wall. This uniform load is comprised of the dead weight of the concrete slab, floor live load, equipment load, weight of interior partitions and weight of exterior curtain walls. For missing panels, convenient values approximately equal to zero are assigned to the thickness and the geometric properties.

4.3 PROPERTIES OF CONNECTOR ELEMENTS

A schematic representation of a connector element is shown in figure (22). Each connector has a shear stiffness denoted by K_v and an axial stiffness denoted by K_a, allowing for two degrees of freedom per connector, that is translation in the x and y directions. Huttelmaier [7] found for a twelve storey shear-wall with four connectors along the horizontal joint per panel and two connector elements along the vertical joint per panel that monolithic behaviour is achieved by shear and axial stiffness values of 14.6×10^7 kN/m per horizontal connector and shear stiffness of 14.6×10^7 kN/m and axial stiffness of 14.6×10^2 kN/m per vertical connector.

Connector stiffnesses used in this investigation were generally more flexible than those mentioned above whch means that full monolithic behaviour was not assumed and stiffness values were selected based on practical results from previous research (4,6,7,9). Table (4) summarizes the connector stiffnesses used in this study.

4.3.1 HORIZONTAL CONNECTOR STIFFNESS

In reference [12], a shear stiffness of 57.58×10^5 kN/m per meter width of wall was found to produce a behaviour within the elastic range. Applying this value to the panel used in this study gives a value of K_v (for horizontal joint) = (57.58×10^5) 3.67 = 21.12×10^6 kN/m (for three connectors per panel).

The axial stiffness (K_a) for the horizontal joints was calculated using an effective modulus of elasticity for a horizontal joint as evaluated by Frank [4] who used the horizontal joint arrangement shown in figure (23) and model for calculation shown in figure (24). The resistance of the model was evaluated by considering only uniaxial stress states and the resistance of the actual joint is considered to consist of the following six main elements.

1) - A layer of grout between floor planks with a Young's modulus approximately half that of the panels. This grout normally extends into the hollow core of the floor planks.
2) - Hollow-core floor planks spanning between adjacent walls.
3) - A layer of drypack concrete with approximately the same stiffness properties as the panel
4) - A layer of bearing pads, usually neoprene or korolath, between the floor planks and the lower wall panel.
5) - Precast panel units immediately above and below the connection area.
6) - The post-tensioning system.

The effective modulus of elasticity for the system was found to be 11.72 kN/m^2 (1700ksi) before creep and as high as 15.51 kN/m^2(2250 ksi) after creep. An approximate average value of $E = 12.2 \times 10^6$ kN/m^2 was used to evaluate the axial stiffness of the horizontal joint in this study. The value of K_a was evaluated by equation (4.1).

$$K_a = \frac{A_p E_{eff}}{L_j} \tag{4.1}$$

where,

A_p = Contact area of panel

= length of panel times thickness

= (3.67) (0.2) = 0.73m^2

E_{eff} = Effective modulus of elasticity of the joint

= 12.2×10^6 kN/m^2

L_j = Thickness of the joint

= 0.28m (11")

$$K_a = \frac{(3.67 \times .2) \times 12.2 \times 10^6}{0.28} = 32 \times 10^6 \text{ kN/m (for three connectors per panel)}$$

4.3.2 VERTICAL CONNECTOR STIFFNESS

Shear stiffness for the vertical connectors was selected according to results found by Pall [6] who recommended a value of 640×10^3 kN/m per connector which makes the total shear stiffness for the vertical joint per panel equal to 1280×10^3kN/m since there are two connectors on the vertical joint per panel. The axial stiffness value was chosen as 14.6×10^3kN/m per connector which is an order of 10 times greater than the limiting value found by Huttelmaier [7]. This value makes the total axial stiffness per panel equal to 29.2×10^3kN/m.

4.4 SEISMIC LOADING

The seismic loading selected as input is represented by the response spectrum for 5% damping of the El Centro California earthquake of May 18, 1940, north-south component [13]. Figure (25) shows the response spectrum curve normalized to peak spectrum acceleration. Table (5) gives the acceleration coefficients for 30 points. The peak ground acceleration is taken as .33g (g = gravitational acceleration) and the peak acceleration for the response spectrum is taken as twice the peak ground acceleration (.66g), [13].

4.5 COMPUTER PROGRAM USED FOR ANALYSIS

The investigation of the structure was performed by the use of the general purpose finite element program Sap IV [14]. The program was developed at the University of California, Berkley and is capable of solving a wide range of three dimensional elastic problems subjected to either static or dynamic loadings.

A subroutine for panel and connector elements was added by Huttlemaier [7]. The inclusion of this subroutine facilitates the solution of large panel structures.

CHAPTER 5

PRESENTATION AND DISCUSSION OF RESULTS

5.1 GENERAL

With respect to progressive collapse, forces in tie and connector elements are calculated for the structure cases detailed in table (1). For a static loading case, results are obtained from equations in chapter 3 as well as finite element analysis, and for the dynamic loading case results are obtained from finite element analysis only. A dynamic loading case is also applied to the structure with panels missing in the interior wall for general structural response and connector element forces. Properties used in the analyses are detailed in tables (2) and (3).

For the static analyses, forces are calculated for assumed tie and connector requirements to withstand progressive collapse. Corresponding forces for the dynamic analysis are also obtained and comparisons will be made to determine the maximum forces for design purposes.

The structure cases (A) and (C) of figure (26) define the limiting conditions for uppermost transverse tie force and horizontal shear force respectively. Case (A) with cantilever height of one panel, gives the maximum tie force in the uppermost transverse tie element in the structure and case (C), with the cantilever height being maximum induces the maximum horizontal shear force and consequently the maximum force in the vertical tie element. The intermediate case, case (B) with the cantilever height of five panels, is selected to estimate the location of the minimum transverse tie requirements and for comparison of force distribution with the finite element analysis.

5.2 TENSILE FORCES IN THE UPPERMOST TRANSVERSE TIE AND CONNECTOR ELEMENT

Table (6) presents the uppermost tie and connector element forces for the three cantilever conditions of figure (26). Since a panel can become ineffective at any level, the cantilever height can vary from one panel depth to (N-1) panels depths. The design force for the uppermost tie element corresponds to case (A), as noted previously.

Examination of table (6) shows that for a cantilever depth of one panel the tensile force in the uppermost transverse tie element, obtained from equation (3.1), is identical to the tensile force in the uppermost vertical connector element, obtained from the finite element analysis. This means that for a cantilever depth of one panel the behaviour is similar for both methods. The force obtained from the dynamic analysis is lower than the results for the static dead load case because the dynamic input load has a horizontal component which makes the loading conditions different.

As the cantilever height increases to the maximum of (N-1) panels the tensile force in the vertical connector (from finite element analysis) corresponding to the uppermost tie element becomes lower than values obtained from equation (3.1). Figures (27) and (28) present axial force distributions for transverse ties and corresponding vertical connector elements along the height of structure for cases (B) and (C) of figure (26). The force distribution for the transverse ties follows a linear pattern corresponding to the strain distribution of figure (15b) but the distribution for the tensile forces of the vertical connectors (from finite element analysis) produces a pattern similar to the distribution of stress in a typical deep beam. This means that whereas the theory for the simple cantilever assumption considers ideal linear strain distribution for cantilevers deeper than one panel, the model for the finite element analysis represents deep beam behaviour. The location of maximum tensile force for vertical connector elements obtained from the finite element analysis for cantilevers deeper than one panel therefore is not at the uppermost element. Consequently, the tensile force in the uppermost tie element, obtained from equation (3.1), will always be greater than or equal to the corresponding value obtained from the finite element analysis. For the design of the uppermost tie element, therefore, the result from equation (3.1) is adequate.

5.3 MAXIMUM FORCE IN TRANSVERSE TIES AND MAXIMUM TENSILE FORCE IN VERTICAL CONNECTOR ELEMENTS AT ANY LEVEL

The maximum tensile force in the transverse tie elements and vertical connector elements, for any cantilever height, will always be smaller than the value for the uppermost level with single panel cantilever of figure (26) case (A). However, determination of the forces at all levels is necessary so that ties are not excessively over-designed as well as to determine the cut-off level for minimum tie requirement. Table (7) and Figure (29) show the maximum values for transverse tie forces using equations (3.2a) and the maximum tensile forces for vertical connector elements from finite element analysis, at any level (x, $n \leq x \leq N$), for any cantilever height. The forces in the transverse ties decrease along the depth of the structure with values less than the minimum requirement of 80 kN at level 10 and lower. However, the finite element analysis produces values which are less than the minimum of 80 kN between levels 10 to 7 and 5 to 1. Values for levels 5 and 6 are higher than the minimum requirements. Design forces for transverse ties will therefore be selected from the finite element analysis and are recorded in table (8).

5.4 VERTICAL TIE FORCES

Vertical ties satisfy three different conditions for stability. Shear friction at horizontal joints is assured by dowel action of vertical ties which also resist suspension of the lower panel and floor loads. These two forces are additive. In addition, vertical ties resist any overturning moments that develop due to eccentricity or horizontal loadings. Since forces in vertical ties are directly related to horizontal shear forces by shear friction action, it is necessary to first determine the maximum horizontal shear force at any level. Table (9) lists the maximum horizontal shear force for the three cantilever mechanisms being considered. The maximum horizontal force occurs at the level immediately above the base of the cantilever for maximum cantilever height (figure 26C). The minimum occurs for minimum cantilever height, figure (26A).

Table (10) lists the total vertical tie force due to shear friction and suspension of lower levels as well as corresponding tensile forces from the finite element analysis. For the finite element analysis for static loading, the tensile force also increases with increasing cantilever depth but the minimum value is approximately 84% of the minimum value from equation (3.6) and the maximum value is 17% greater than the maximum values from equation (3.6). Results for the dynamic analysis also increase with increasing cantilever depth but values are in the range of 26% to 38% of the static analysis.

Overturning moments induced by cantilever action due to static loading is resisted by gravity forces since the adjacent uniform stack of panels are equally loaded. Overturning moment to be resisted by the vertical ties, is therefore due to horizontal loading or as in this study due to dynamic loading. For the dynamic analysis the maximum overturning moment occurs for the structure case with all panels in place as shown in figure (37) and section 5.7.7. This overturning moment is resisted by gravity loads resulting in a net stabilizing moment (table 11).

Design force for the vertical tie will therefore be the maximum value from table (10). This value represents the total force per panel. Thus, with three vertical ties per panel the force per tie will be 1/3 this maximum value.

5.5 FORCE IN THE COMPRESSION BLOCK

The compression component due to the cantilever moment couple, as outlined in section 3.4.3, can develop at the base of any panel since any panel can be the lowest level in a cantilever.

Table (9) lists the compression components for the three structure cases of figure (26). The maximum compression force occurs at the base of the lowest panel for the maximum cantilever (figure 26, case C) from finite element analysis. As stated earlier this force is transferred to adjacent stack of panels through grout over the lower quarter of the panel.

5.6 VERTICAL SHEAR FORCE

The vertical shear force induced by cantilever action, as stated in chapter (3), is equal for each panel for constant panel height and constant load on panels and is equal to the load per panel. Figure (30) shows for the finite element analysis (for static loading) the maximum possible vertical shear force for each panel was found to be minimum at the uppermost panel and maximum at the lowest panel for maximum cantilever height. Values for the uppermost panel are identical for both analyses but the highest value from finite element analysis is approximately 380% of the value obtained from cantilever assumption (equation 3.8) for the lowest panel. Vertical shear connectors should therefore be proportioned according to the result of the finite element analysis (Table 12).

5.7 DYNAMIC ANALYSIS

5.7.1 GENERAL

The dynamic analysis was performed for panels missing in both interior and exterior walls as well as for a uniform structure with all panels being in place. Values for fundamental period, maximum deflection, base shears, base overturning moments and maximum connector forces for cases with ineffective panels in both walls are obtained and normalized to maximum values for the structure with all panels in place. The results obtained employed the first twelve modes and the data are plotted in figures (31 to 38).

5.7.2 EFFECT OF MISSING PANEL ON MAXIMUM AXIAL FORCE FOR VERTICAL CONNECTORS

Figure (31) represents the ratio of the maximum axial force (F_{va}) for vertical connectors for the structure with a missing panel, to the maximum axial force ($\bar{F}_{va}$)for the vertical connectors for the structure with all panels in place.

For panels missing in the interior wall the ratio ranges between 1.0 to 0.5 for panels missing in the upper region of the wall (between levels 5 to 12). The effect of panels missing at levels lower than 5 is not significant and values are approximately equal to those of the structure with all panels in place. For the exterior wall the effect is opposite.

Maximum force for panels missing above level 5 increases to a maximum of 150% of the maximum force for the wall with all panels in place and to a maximum of 140% for panel missing at the lowest level. For panel missing at level 3 the increase is about 10%.

5.7.3 EFFECT OF MISSING PANEL ON MAXIMUM SHEAR FORCE FOR VERTICAL CONNECTORS

Figure (32) represents the ratio of the maximum shear force (F_{vv}), for vertical connectors for the structure with a missing panel, to the maximum shear force ($\bar{F}_{vv}$) for the vertical connectors for the structure with all panels in place.

For panels missing in the interior wall there is no significant change in the maximum shear force from the wall with all panels in place. The maximum deviation is approximately 4% for panels missing at level 8 and level 1. However, for panels missing in the exterior wall, the maximum increase in shear force is about 240% for the panel missing at the lowest level. The difference decreases almost linearly from 200% for level 4 to zero for level 11.

5.7.4 EFFECT OF MISSING PANEL ON MAXIMUM AXIAL FORCE FOR HORIZONTAL CONNECTORS

Figure (33) represents the ratio of the maximum axial force (F_{ha}), for horizontal connectors for the structure with a missing panel, to the maximum axial force ($\bar{F}_{ha}$) for the horizontal connectors for the structure with all panels in place.

The variation in maximum forces for panels missing in both walls is approximately similar for panels missing above level 3 with a maximum decrease of 25%. For the panel missing at the lowest level however, the increase in force for the exterior wall is 25% greater whereas there is no difference for the interior wall.

5.7.5 EFFECT OF MISSING PANEL ON MAXIMUM SHEAR FORCE FOR HORIZONTAL CONNECTORS

Figure (34) represents the ratio of the maximum shear force (F_{hv}), for horizontal connectors for the structure with a missing panel, to the maximum shear force ($\overline{F}'_{hv}$) for the horizontal connectors for the structure with all panels in place.

The effect for panels missing in both walls is similar except for the panel missing at the lowest level. There is a decrease in force of 25% for panels missing at level 6 in both exterior and interior walls but there is an increase of 50% for the exterior wall and 25% for the interior wall with the panel missing at the lowest level.

5.7.6. EFFECT OF MISSING PANEL ON FUNDAMENTAL PERIOD AND MAXIMUM DEFLECTION OF WALL

Figures (35) and (36) show the effect of a missing panel on the flexibility of the structure.

Figure (35) represents the ratio of the fundamental period (T'_s), for the structure with a missing panel, to the fundamental period ($\overline{T}_s$) for the structure with all panels in place and figure (36) represents the ratio of the maximum deflection (Δ_s)', for the structure with a missing panel to the maximum deflection ($\overline{\Delta}_s$) for the structure with all panels in place. The effect in both figures (35) and (36) are similar. There is a 5% increase in the fundamental period and maximum deflection of the structure for a panel missing at level 3 in the interior wall. For panels missing in the exterior wall, fundamental period and maximum deflection increase from 10% for a panel missing at level 7 to a maximum of 60% for a panel missing at the lowest level. There is no change in fundamental period and maximum deflection for the panel missing at level 11.

5.7.7 EFFECT OF MISSING PANEL ON BASE OVERTURNING MOMENT

Figure (37) represents the ratio of the base overturning moment (M_o), of the structure with a missing panel, to the base overturning moment ($\overline{M}_o$) of the structure with all panels in place. For the interior wall the maximum reduction in base overturning moment is 15% for a panel missing at level 6 and for the exterior wall the maximum reduction in base overturning moment is 40% for a panel missing at level 4.

5.7.8 EFFECT OF MISSING PANEL ON BASE SHEAR

Figure (38) represents the ratio of the base shear (V_s) of the structure with a missing panel, to base shear ($\overline{V}_s$) for the structure with all panels in place. For the interior wall the maximum reduction in base shear is 15% for a panel missing at level 6 and for the interior wall the maximum reduction is 40% for a panel missing at level 4.

CHAPTER 6

CONCLUSIONS

6.1 STATIC ANALYSIS

An analysis of a twelve storey panelized shear wall was performed with respect to progressive collapse. The analysis was carried out by a finite element method for both static and dynamic loadings and the results were compared to the results of a cantilever assumption method as developed in reference [2]. To represent conditions for progressive collapse, panels were assumed ineffective as outlined in table (1).

To withstand progressive collapse tie elements were assumed for the cantilever method and lumped connector properties in the form of shear and axial stiffnesses were assumed for the finite element analysis. Forces developed in the uppermost transverse tie elements were found to be identical for both methods of calculations for static loading (table 6) for cantilever of one panel but values from the finite element analysis were found to be more critical for tie elements at lower levels (table 7 and figure 29). Forces for the dynamic analysis were found to be lower than values from the static analysis (table 6). Horizontal shear forces of horizontal joints were found to be identical at the uppermost level but the finite element analysis results were found to be more critical for panels at lower levels with a maximum at the lowest level for a maximum cantilever height (table 9). Vertical tie forces are directly related to horizontal shear force and the maximum value was found at the lowest level for the highest cantilever height (table 10). The force in the compression block, for each panel, created by cantilever moment, is equal to the horizontal shear force for that panel and is also given in table (9). Results for vertical shear per panel are given in figure (30).

The maximum value from finite element analysis is approximately 380% the value from the simple cantilever assumption.

6.2 DYNAMIC ANALYSIS

A dynamic analysis was carried out for ground acceleration represented by a response spectrum for 5% damping and was performed by a finite element analysis. Connector forces and general structural responses were obtained for the wall with all panels in place as well as with the wall with panels ineffective in both the interior and exterior stacks (table 1). Compared to the wall with all panels in place significant effects were found on axial and shear forces of the vertical connectors for panels missing in the exterior stack. For axial force an increase of 50% was found (figures 31 and 32) and for shear force an increase of 240% was found. Effect for panels missing in the interior stack was not significant. The effect on forces in the horizontal connectors were similar for panels missing in both stacks (figure 33 and 34). With respect to structural response of the structure (figures 35 to 38) panels missing in the exterior stack had greater effect. Fundamental period and maximum deflection for panels missing in the exterior stack increased by 60% whereas there was no significant change when panels were missing in the interior stack.

6.3 CLOSING REMARKS

The present study analyses the response of a panelized shear wall in the elastic range. That is, the panel and the joints are proportioned in such a way that the stresses in these elements do not exceed the elastic limits under a given seismic loading condition. However, in practical design it is important to know the elasto-plastic behaviour of the panel wall system in order to achieve economy in design of the elements. Future study may therefore be directed to the elasto-plastic response analysis of a panelized shear-wall with respect to progressive as well as total collapse.

TABLE 1 - PROGRAM OF STUDY

(Reference Figure 1)

ITEM	ANALYSIS	CONDITION OF STRUCTURE	
		INTERIOR WALL	EXTERIOR WALL
A	SIMPLE CANTILEVER ASSUMPTION BASED ON EQUATIONS OF CHAPTER 3 (STATIC LOADING)	---------	PANELS MISSING AT ALL LEVELS
B	FINITE ELEMENT ANALYSIS (STATIC LOADING)	--------	SAME AS ABOVE
C	FINITE ELEMENT ANALYSIS (SEISMIC LOADING)	PANEL MISSING BETWEEN LEVELS 0 & 1;2 & 3; 5 & 6; AND 10 & 11	PANEL MISSING BETWEEN LEVELS 0 & 1; 3 & 4; 6 & 7; AND 10 & 11

TABLE 2

DATA FOR EQUATIONS IN CHAPTER 3

(Static Analysis)

W_s	=	TOTAL LOAD ON WALL (DEAD AND LIVE LOADS FROM SLAB & WEIGHT OF WALL)
	=	78.66 kN/m WIDTH OF WALL
l_d	=	LENGTH OF CANTILEVER CREATED
	=	3.67m
h_s	=	HEIGHT OF STOREY
	=	2.97m
N	=	NUMBER OF STOREYS IN WALL
	=	12
n	=	STOREY NUMBER THAT IS CONSIDERED INEFFECTIVE

TABLE 3

DATA FOR FINITE ELEMENT ANALYSIS

(Static And Dynamic Analyses)

		Joint Stiffnesses are detailed in Table 4
W_1		(For static analysis) = total load on wall (Dead and Live Loads from slab & weight of wall)
	=	78.66 kN/m width of wall
W_2		(For dyanmic analysis) = total dead load on wall. (Dead load from slab & weight of panel).
	=	68.66 kN/m width of wall
L	=	Length of panel
	=	3.67 m
h	=	Height of panel
	=	2.97m

TABLE 4

SUMMARY OF JOINT STIFFNESSES

TYPE OF JOINTS	SHEAR STIFFNESS K_v =kN/m	AXIAL STIFFNESS K_a =kN/m
HORIZONTAL JOINT (TOTAL FOR 3 CONNECTOR ELEMENTS/ PANEL)	21.12×10^6	32.00×10^6
VERTICAL JOINT (TOTAL FOR 2 CONNECTOR ELEMENTS/PANEL)	12.80×10^5	29.20×10^3

TABLE 5

SPECTRUM DATA FOR SEISMIC INPUT

POINT NO.	PERIOD SEC.	SPECTRAL ACCELERATION COEFFICIENTS m/sec^2/ m/sec^2
1	0	.5
2	.05	.6
3	.10	.7
4	.15	.8
5	.20	.9
6	.25	1.0
7	.30	.876
8	.35	.780
9	.40	.706
10	.45	.646
11	.50	.597
12	.60	.521
13	.70	.464
14	.80	.420
15	.90	.384
16	1.00	.355
17	1.10	.331
18	1.20	.310
19	1.30	.292
20	1.40	.276
21	1.50	.262
22	1.80	.228
23	2.10	.204
24	2.40	.184
25	2.70	.169
26	3.00	.156
27	3.30	.145
28	3.60	.136
29	3.90	.128
30	4.20	.121

TABLE 6

TENSILE FORCES IN UPPERMOST TRANSVERSE TIE ELEMENT AND UPPERMOST VERTICAL CONNECTOR ELEMENT

STRUCTURE		α_r FROM FIG. 16	TENSILE FORCES IN UPPERMOST ELEMENTS (kN)				
			TRANSVERSE TIE ELEMENT F_r		VERTICAL CONNECTOR ELEMENT (FINITE ELEMENT ANALYSIS)		
					STATIC LOAD		DYNAMIC ANALYSIS
CASE	PANEL MISSING BETWEEN LEVELS		DEAD LOAD	DEAD & LIVE LOADS	DEAD LOAD	DEAD & LIVE LOAD	DEAD LOAD
A	10 & 11	1.00	155.6	178.3	155.6	178.2	132
B	6 & 7	0.45	70.0	80.3	69.8	80.0	122
C	0 & 1	0.22	34.2	39.2	5.3	6.1	43

$$F_r = \alpha_r \frac{W_s\,l^2\,d}{2\,h_s} \qquad \text{Equation (3.1)}$$

DEAD LOAD = 68.66 kN/m width of wall

DEAD & LIVE LOADS = 78.66 kN/m width of wall

TABLE 7

MAXIMUM TENSILE FORCES IN TRANSVERSE TIE ELEMENTS AND VERTICAL CONNECTOR ELEMENTS AT ANY LEVEL

(Static Analysis, Dead & Live Loads)

LEVEL	α_x FROM FIG.17	MAXIMUM TENSILE FORCE AT ANY LEVEL kN	
		TRANSVERSE TIE ELEMENT F_x	VERTICAL CONNECTOR ELEMENT (FINITE ELEMENT ANALYSIS)
12	1.00	178.3	178.2
11	0.45	80.3	142.1
10	0.29	52.0	73.3
9	0.21	37.4	71.0
8	0.17	30.3	72.0
7	0.14	25.0	78.0
6	0.12	21.4	89.0
5	0.10	17.8	89.4
4	0.10	17.8	69.0
3	0.10	17.8	8.0
2	0.10	17.8	0
1	0.10	17.8	0

$$F_{x_{max}} = \alpha_x \frac{W_s l^2 d}{2 h_s}$$ Equation (3.2a)

DEAD & LIVE LOADS = 78.66 kN/m width of wall

TABLE 8

DESIGN FORCES FOR TRANSVERSE TIES

LEVELS	DESIGN FORCE FOR TRANSVERSE TIES kN
12	178.3
11	142.1
10 to 7	80.0
6	89.0
5	89.5
3 to 2	80.0

TABLE 9

SHEAR FORCE IN THE LOWEST HORIZONTAL JOINT IN CANTILEVERS

A, B & C (fig. 26)

STRUCTURE		β_x FROM FIG. 19	TOTAL SHEAR FORCE IN LOWEST HORIZONTAL JOINT IN CANTILEVER (kN)				
			V_{h_x}		3 HORIZONTAL CONNECTOR ELEMENTS (FINITE ELEMENT ANALYSIS)		
					STATIC LOAD		DYNAMIC ANALYSIS
CASE	PANEL MISSING BETWEEN LEVELS		DEAD LOAD	DEAD & LIVE LOAD	DEAD LOAD	DEAD & LIVE LOAD	DEAD LOAD
A	10 & 11	1.0	155.6	178.3	155.6	178.3	177.0
B	6 & 7	1.39	216.3	247.8	219.1	251.0	200.0
C	0 & 1	1.45	225.6	258.5	326.9	374.5	247.0

$$V_{hx} = \beta_x \frac{W_s L_d^2}{2 h_s} \qquad \text{Equation (3.3)}$$

DEAD LOAD = 68.66 kN/m width of wall

DEAD & LIVE LOADS = 78.66 kN/m width of wall

TABLE 10

AXIAL FORCE IN THE LOWEST HORIZONTAL JOINT IN CANTILEVERS A,B & C (fig. 26)

STRUCTURE			TOTAL AXIAL FORCE IN LOWEST HORIZONTAL JOINT IN CANTILEVERS A B C (fig.26)(kN)				
			3 VERTICAL TIE ELEMENTS		3 HORIZONTAL CONNECTOR ELEMENTS (FINITE ELEMENT ANALYSIS)		
CASE	PANEL MISSING BETWEEN LEVELS	V_{hx} FROM TABLE 9	T_{vf} DEAD & LIVE LOADS	T_x DEAD & LIVE LOADS	T_{vf} + T_x	(STATIC) DEAD & LIVE LOADS	DYNAMIC DEAD LOAD
A	10 & 11	178.3	178.3	288.7	467.0	391.0	101.0
B	6 & 7	247.8	247.8	288.7	536.0	535.0	195.0
C	0 & 1	258.5	258.5	288.7	553.3	646.0	247.0

$$T_{vf} = \frac{V_{hx}}{\mu} ; \quad \mu = 1.0 \qquad \text{Equation (3.4)}$$

$$T_x = W_s L_d \qquad \text{Equation (3.5)}$$

TABLE 11

NET BASE RESISTING MOMENT (DYNAMIC ANALYSIS)

NO PANELS MISSING

LOAD	WEIGHT kN	LEVER ARM m	OVERTURNING MOMENT kN-m
GRAVITY (DEAD)	288.7 x 3 x 12 = 10393	$\frac{3.67 \times 3}{2}$ = 5.51	- 57,215
DYNAMIC	-----	-----	44,100
NET RESISTING MOMENT = 13,115 kn-m			

TABLE 12

DESIGN SHEAR FORCE FOR VERTICAL CONNECTORS

(RESULTS FROM FINITE ELEMENT ANALYSIS, STATIC DEAD & LIVE LOAD)

FLOOR LEVEL	VERTICAL CONNECTOR SHEAR FORCE kN
12	288.7
11	360.0
10	450.0
9	540.0
8	600.0
7	670.0
6	750.0
5	860.0
4	730.0
3	1000.0
2	1100.0

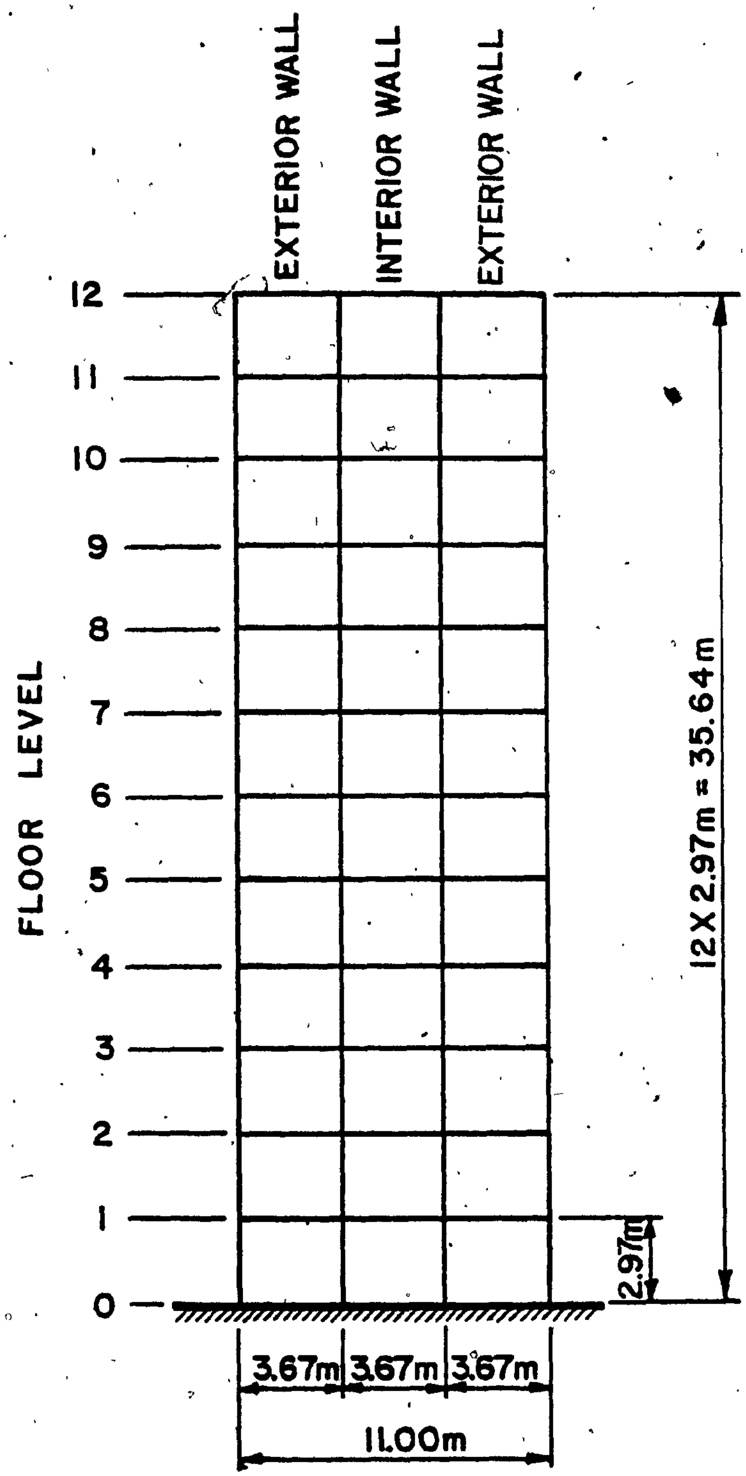


FIGURE (1) - PANEL WALL STRUCTURE

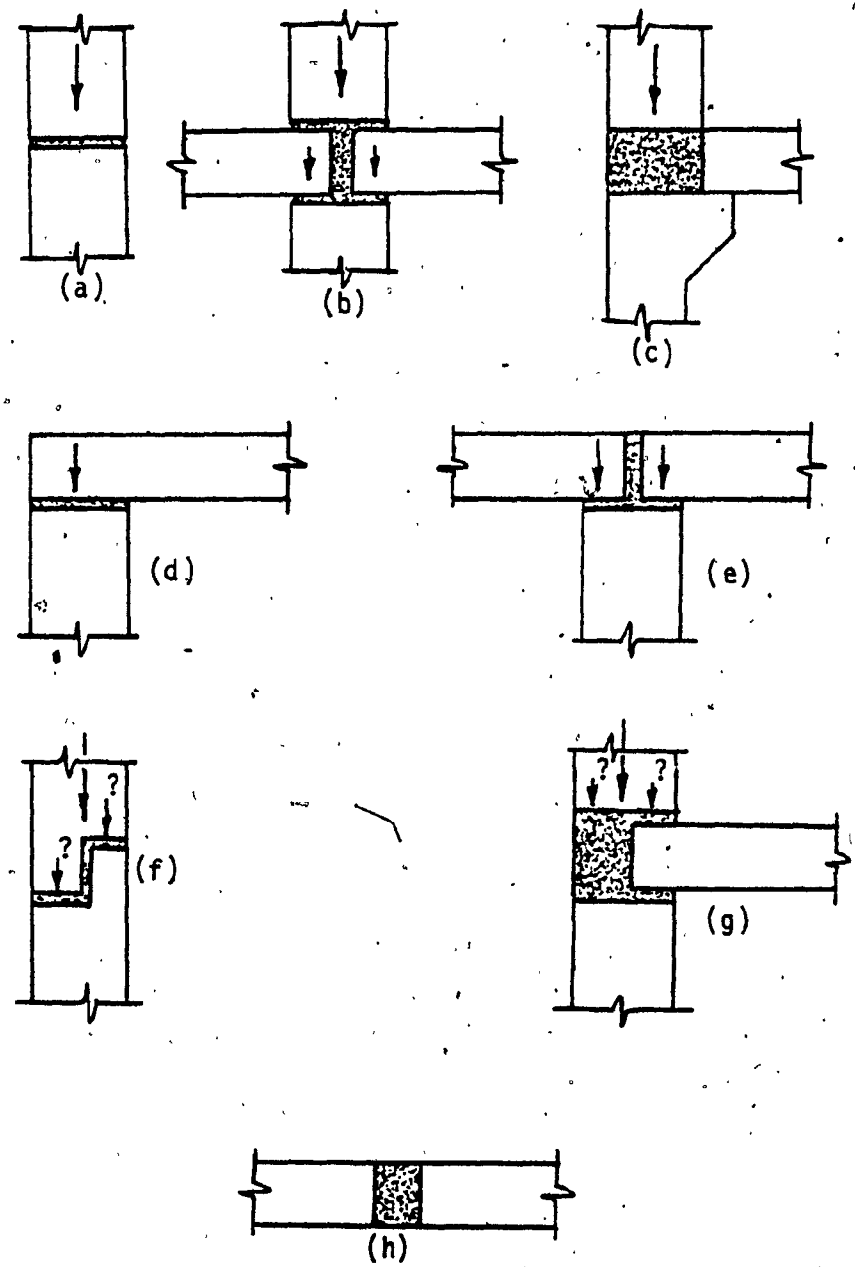


FIGURE (2) - HORIZONTAL JOINTS - VERTICAL SECTIONS SHOWING LOAD TRANSFER FROM UPPER STOREYS, (REF. 1 & 5)

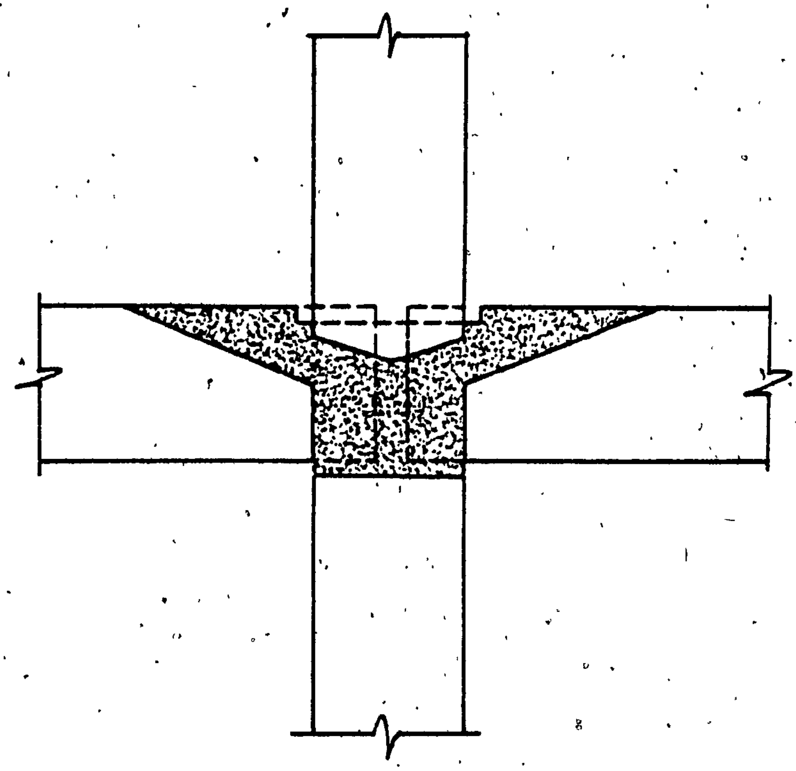
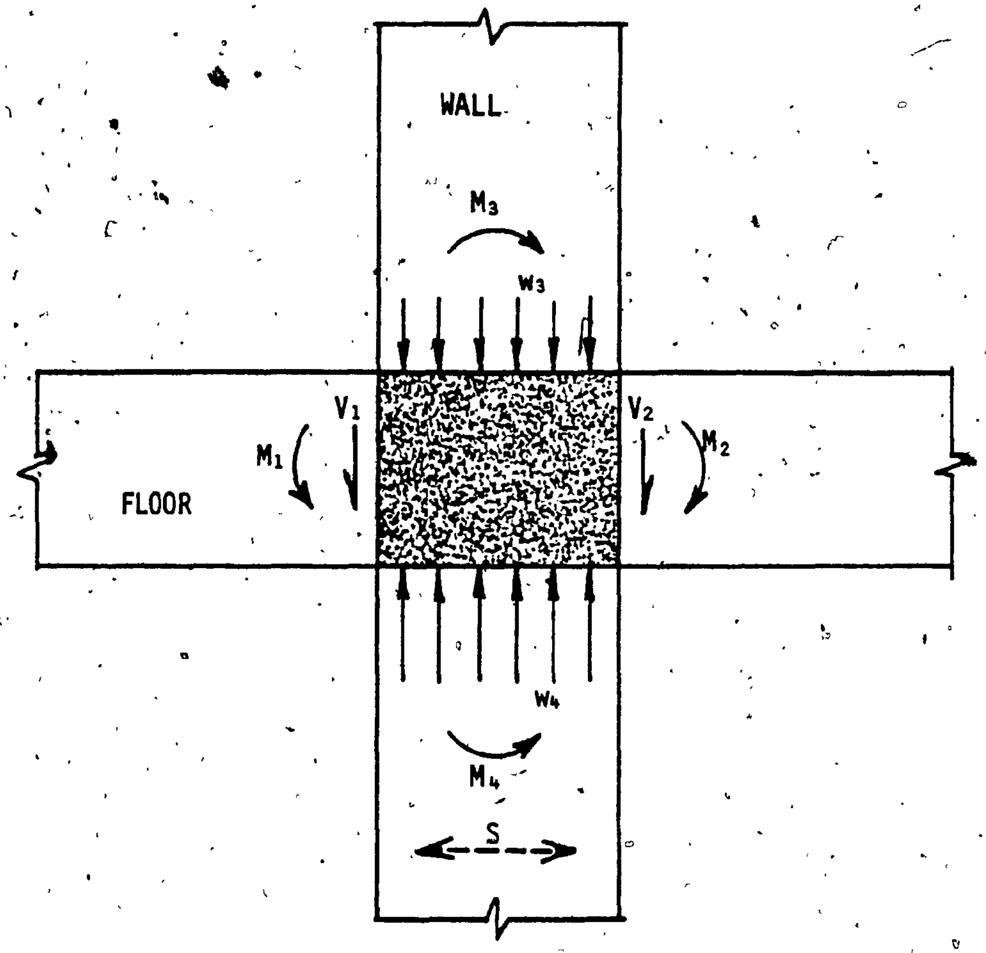


FIGURE (3) - PRACTICAL HORIZONTAL JOINT DETAIL
(REF. 1)



W_3 = GRAVITY LOAD FROM ABOVE

M_3 = MOMENT DUE TO ECCENTRICITY OF W_3

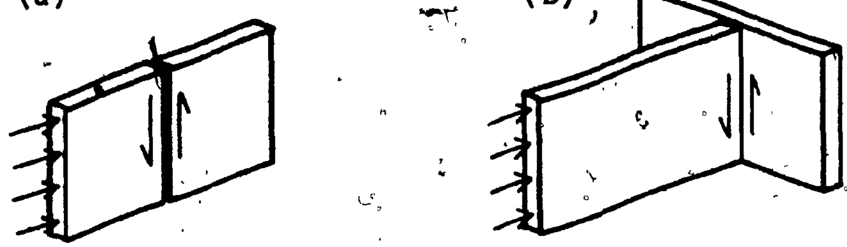
V_1, V_2 = GRAVITY LOAD FROM FLOOR SLABS

W_4, M_4 = REACTIONS

S = TENSILE SPLITTING STRESS IN LOWER PANEL

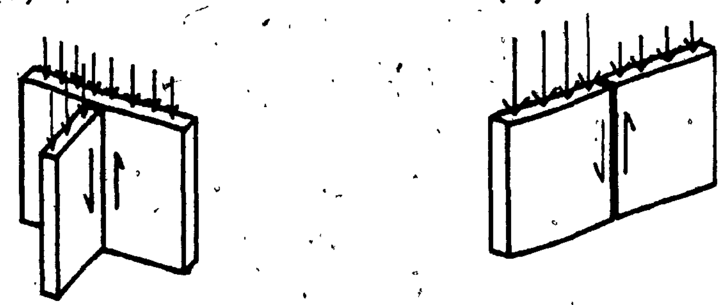
FIGURE (4) - POTENTIAL FORCES AT HORIZONTAL JOINT DUE TO GRAVITY LOADS, (REF. 5)

(a) (b)



(a), (b) TANGENTIAL FORCES DUE TO LATERAL LOAD

(c) (d)

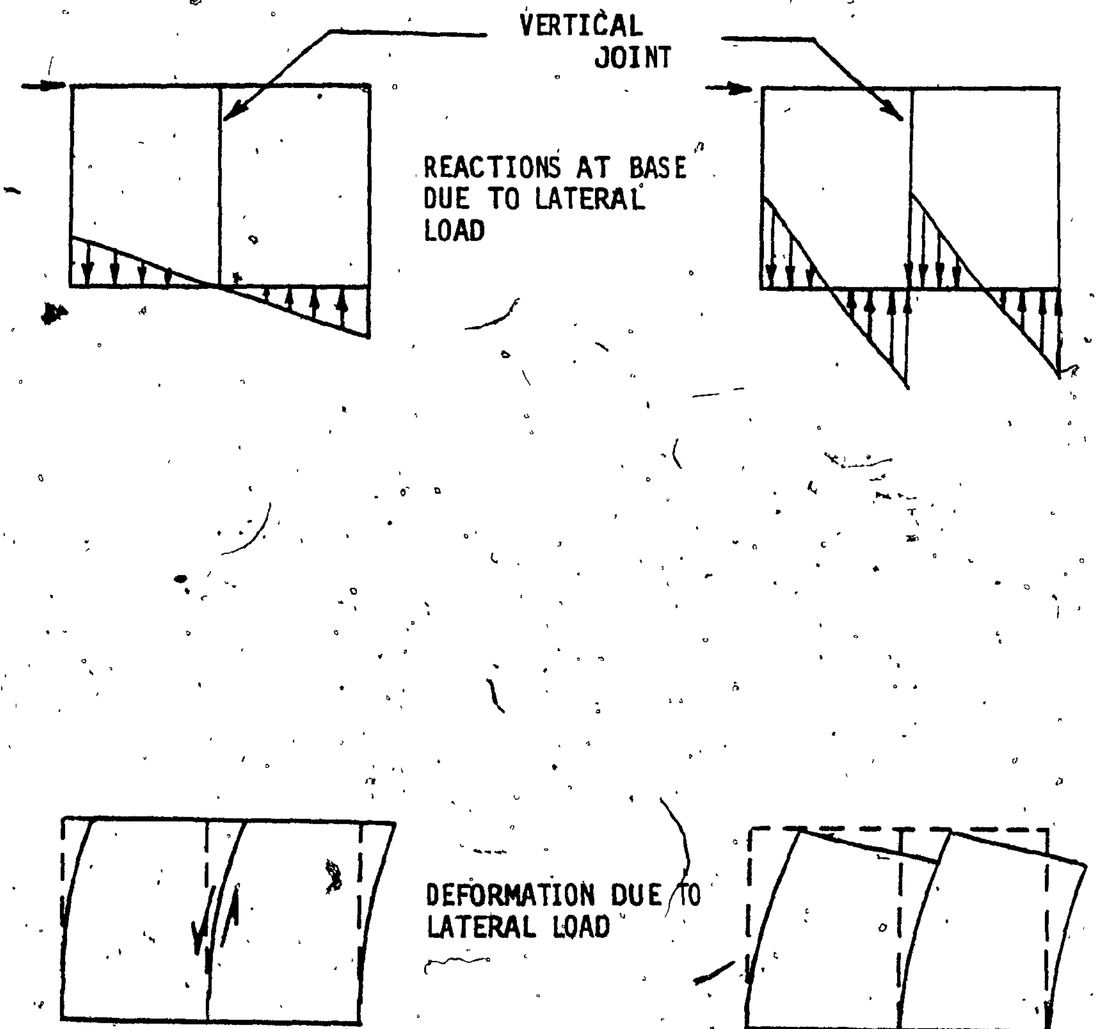


(c), (d) TANGENTIAL FORCES DUE TO DIFFERENTIAL GRAVITY LOAD

(e) (f)

(e), (f) HORIZONTAL TENSILE FORCE DUE TO BUCKLING OF ADJACENT WALLS

FIGURE (5) - FORCES IN VERTICAL JOINTS (REF. 1 & 5)



a) STIFF VERTICAL JOINT

b) SOFT VERTICAL JOINT

FIGURE (6) - EFFECT OF STIFFNESS OF VERTICAL JOINT ON SHEAR-WALL BEHAVIOUR, (ref.5)

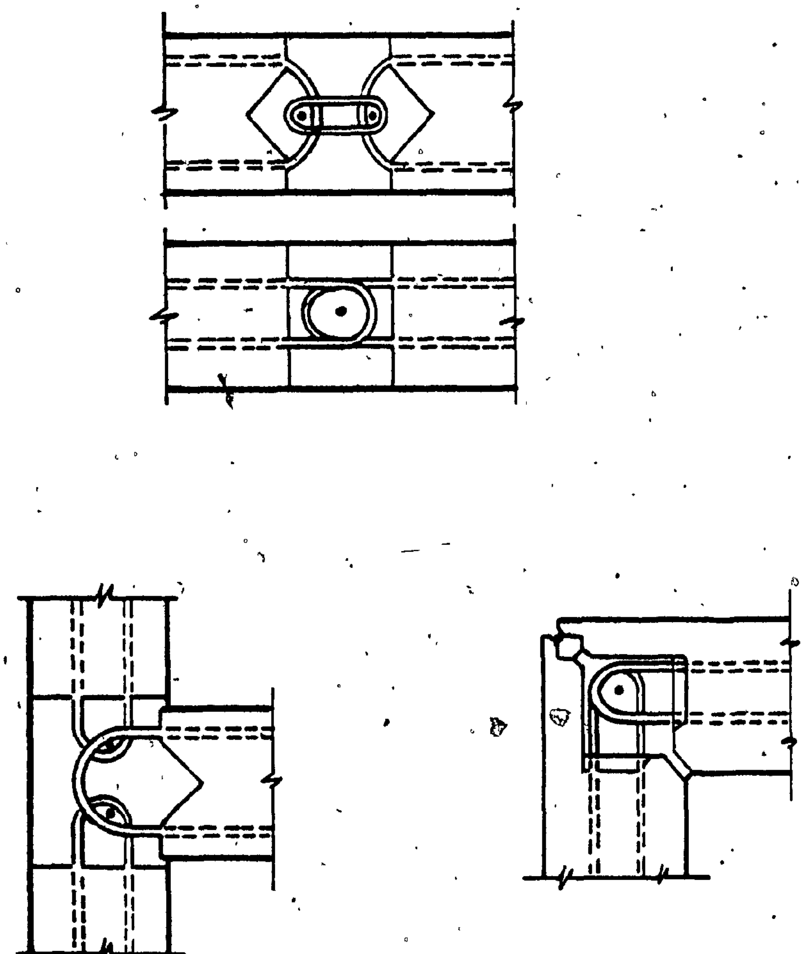
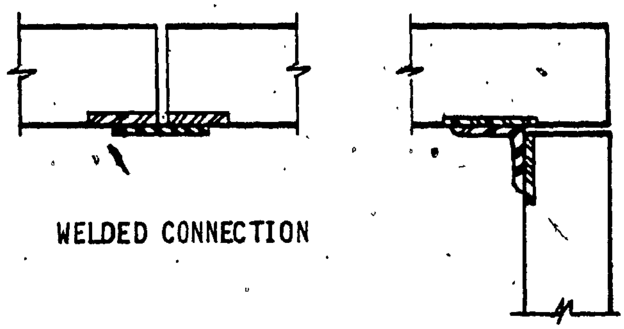
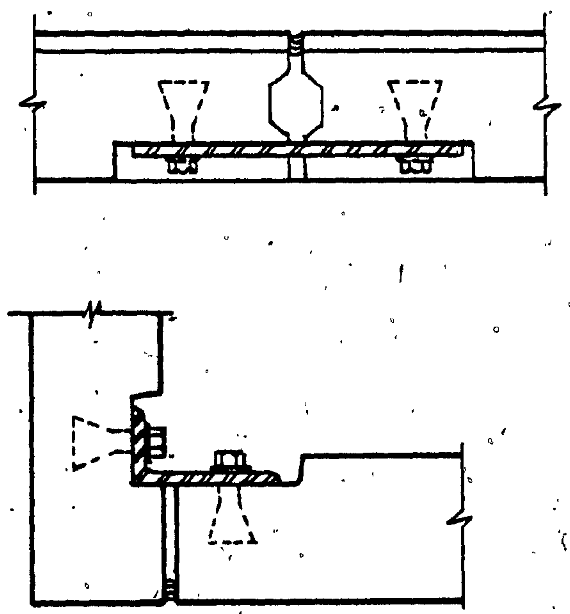


FIGURE (7) - WET REINFORCED VERTICAL JOINTS,(REF. 5)



WELDED CONNECTION



BOLTED CONNECTION

FIGURE (8) - DRY VERTICAL JOINTS, (REF. 5)

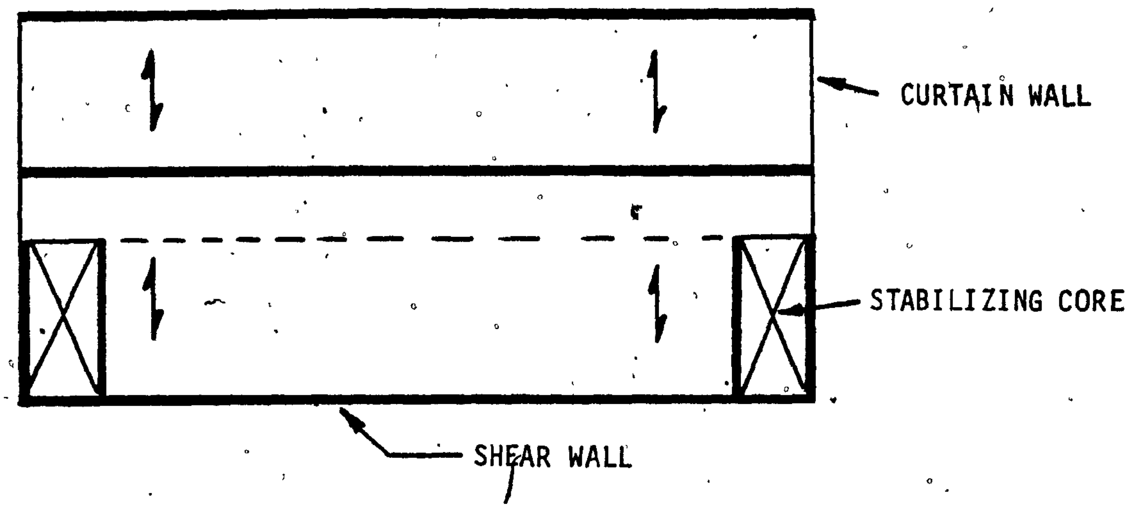


FIGURE (9) - LONGITUDINAL WALL SYSTEM, (REF.2)

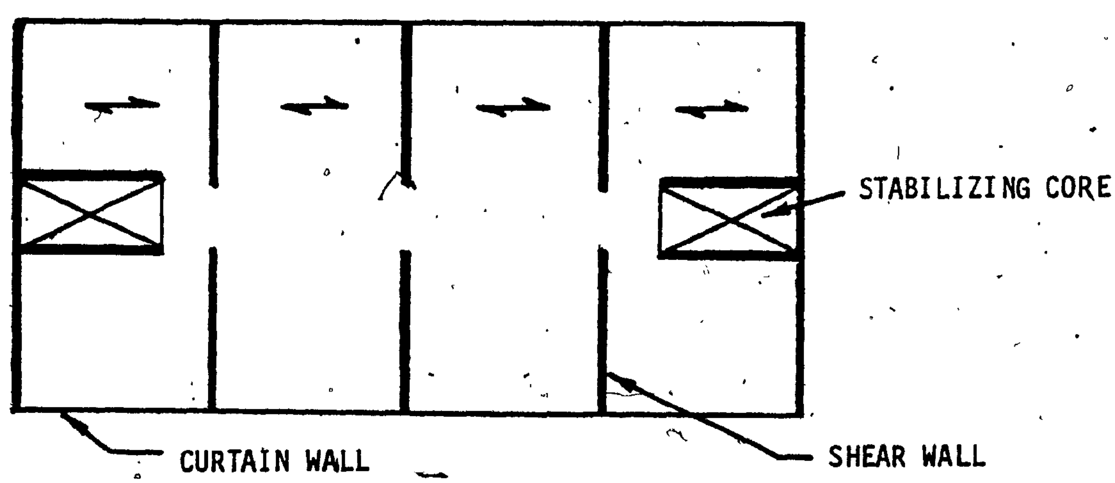


FIGURE (10) - CROSS-WALL SYSTEM, (REF. 2)

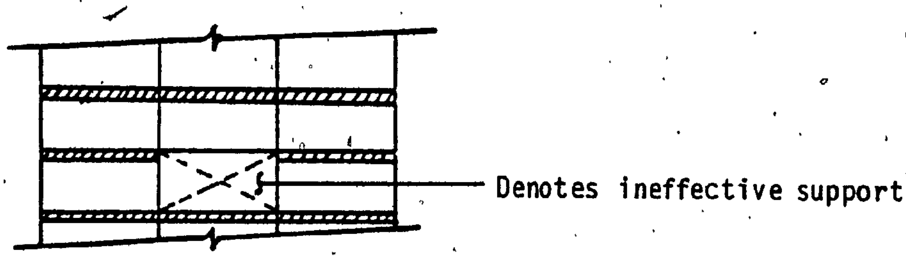


FIGURE (11) - BEAM ACTION (REF. 2)

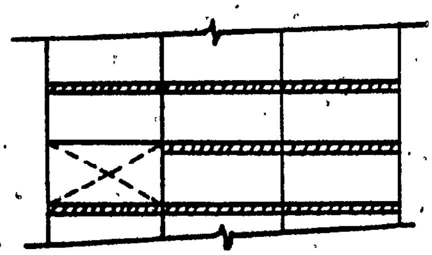


FIGURE (12) - CANTILEVER ACTION (REF. 2)

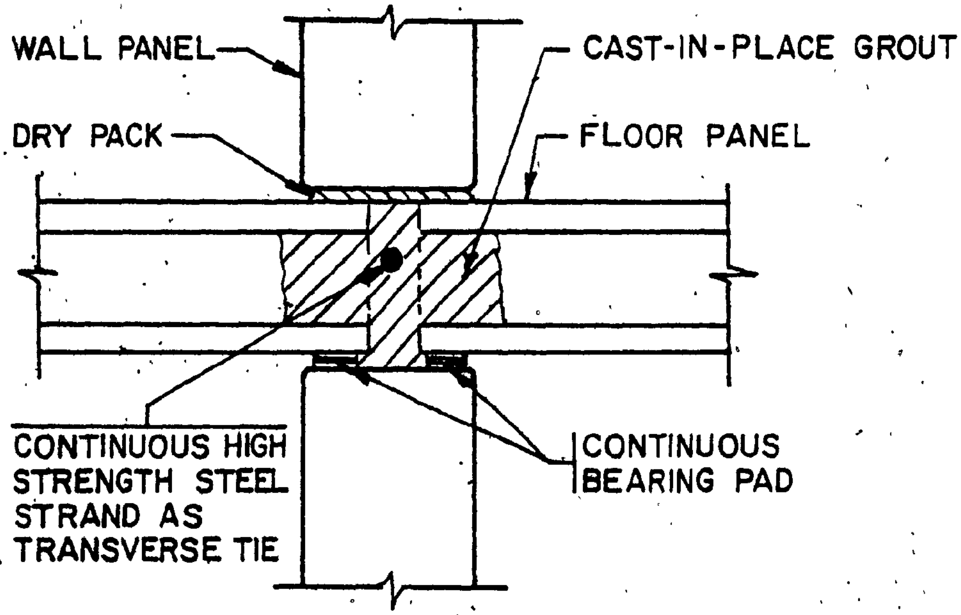


FIGURE (13) - TRANSVERSE TIE ARRANGEMENT, (REF. 2)

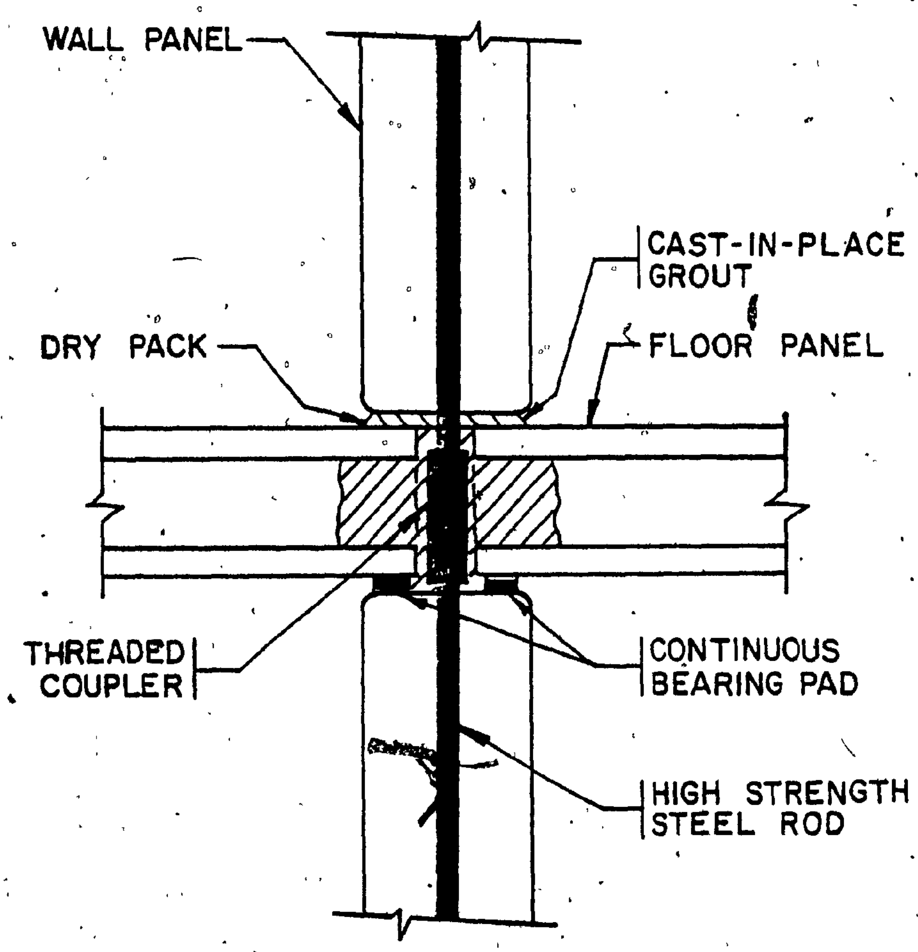
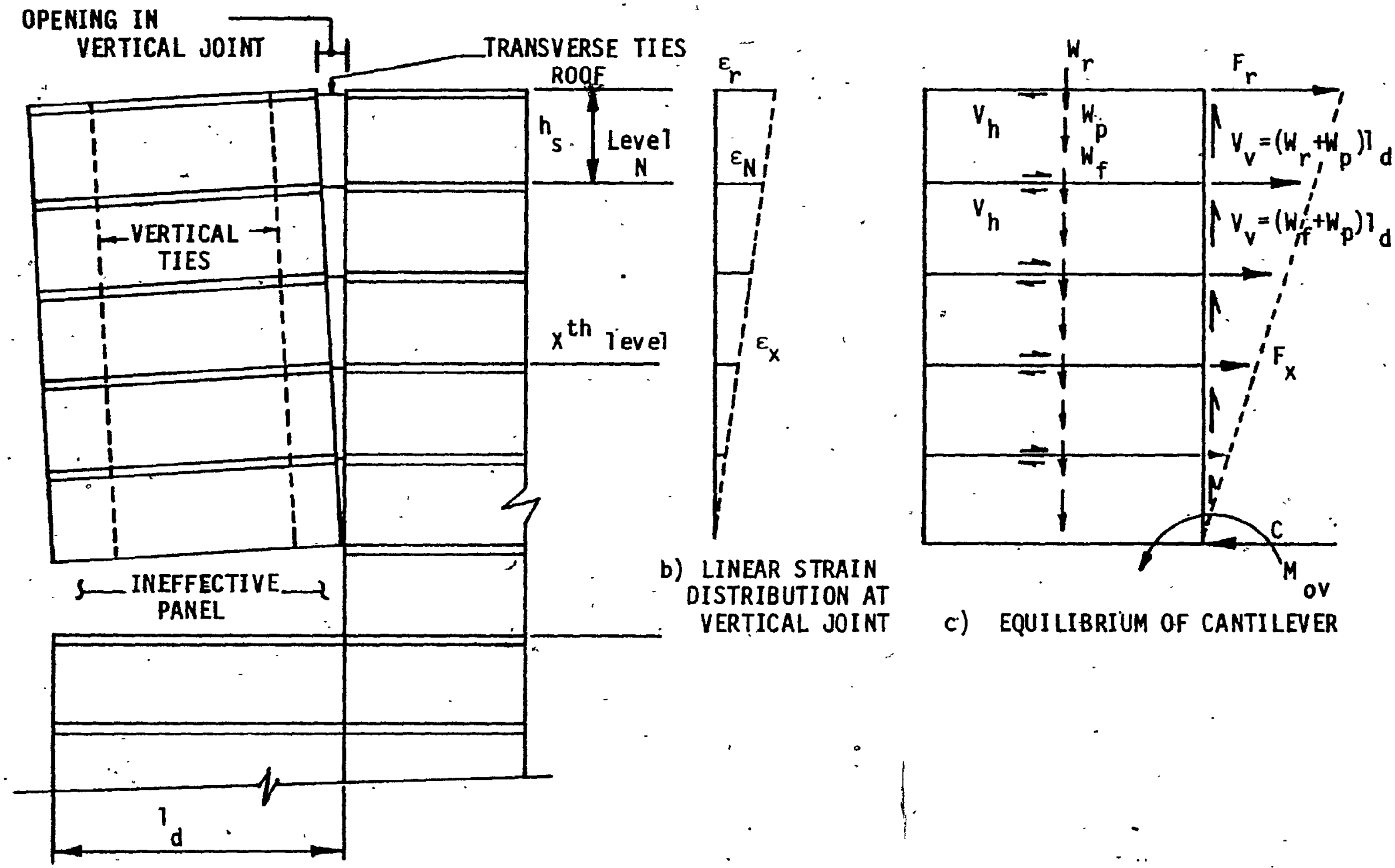


FIGURE (14) - VERTICAL TIE ARRANGEMENT, (REF. 2)



a) CANTILEVER MECHANISM IN STABLE EQUILIBRIUM

b) LINEAR STRAIN DISTRIBUTION AT VERTICAL JOINT

c) EQUILIBRIUM OF CANTILEVER

FIGURE (15) - IDEALIZED CANTILEVER MECHANISM, (REF. 2)

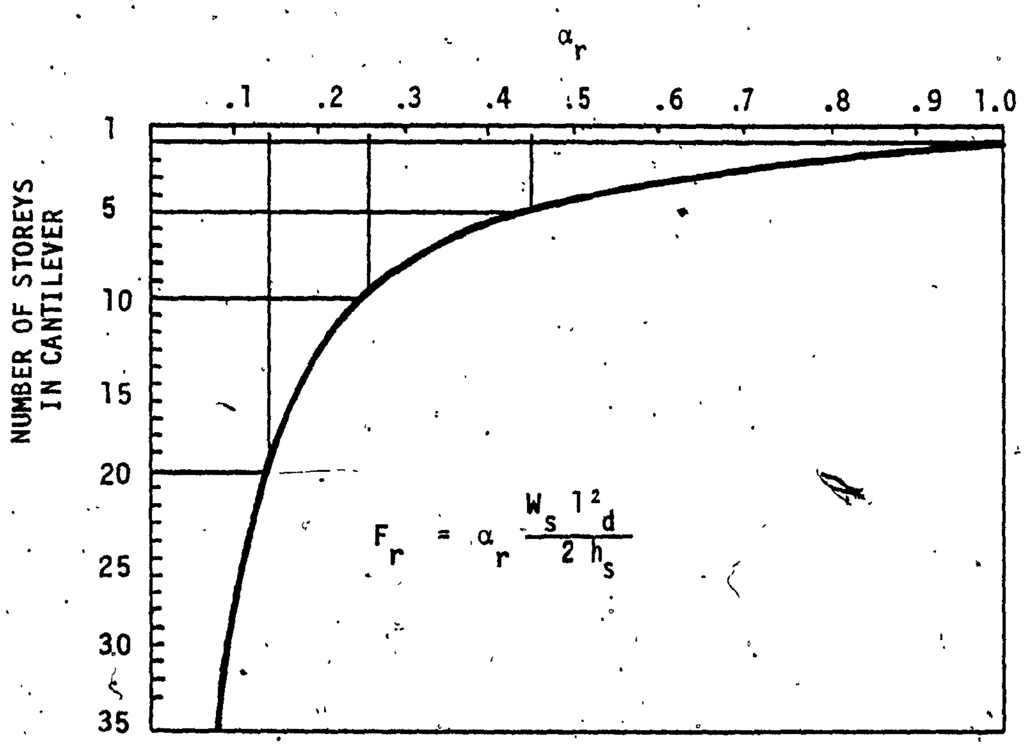


FIGURE (16) - FORCE "F_r" AT UPPERMOST TRANSVERSE TIE FOR VARYING CANTILEVER HEIGHTS, (REF.2)

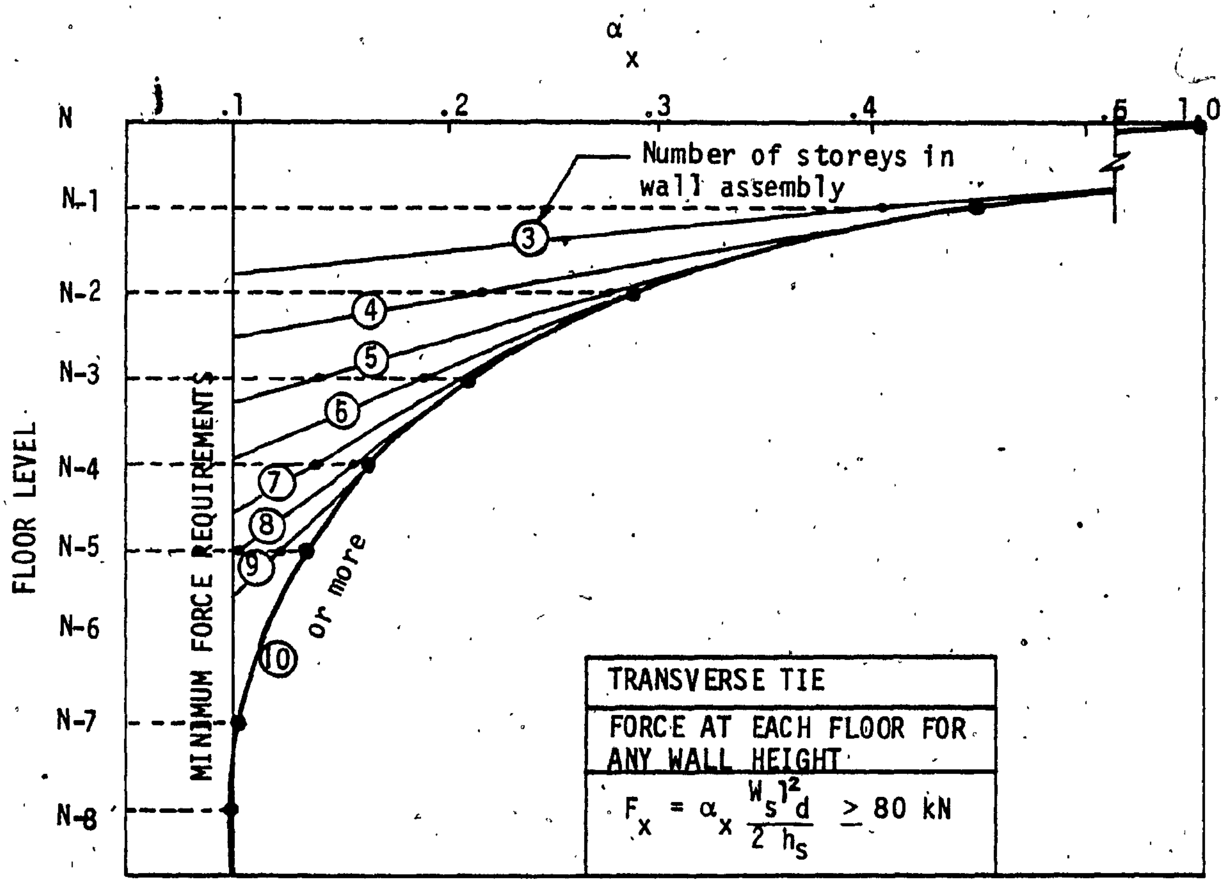


FIGURE (17) - DESIGN CHART FOR TRANSVERSE TIE FORCE AT ANY FLOOR LEVEL (REF.2)

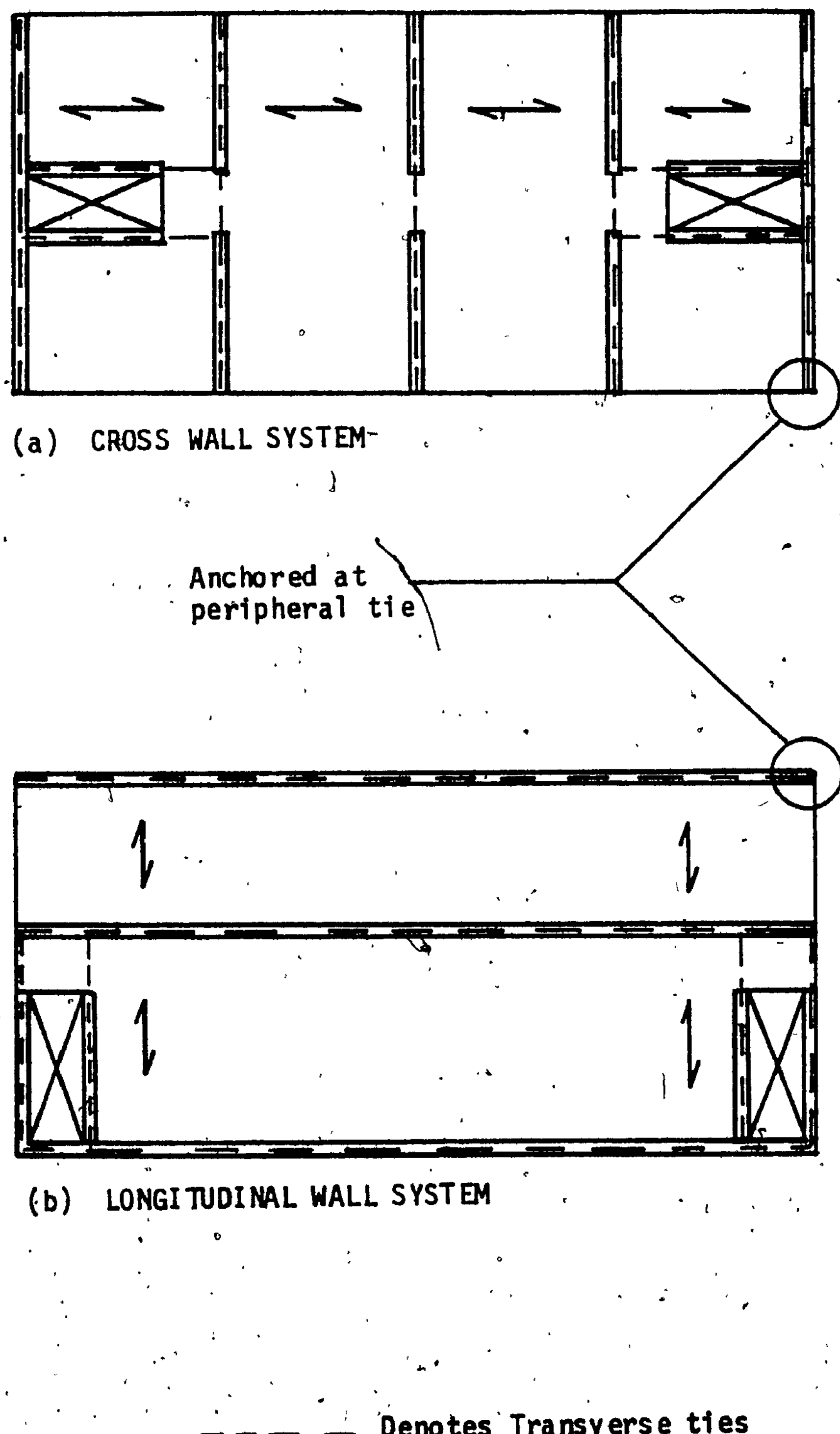
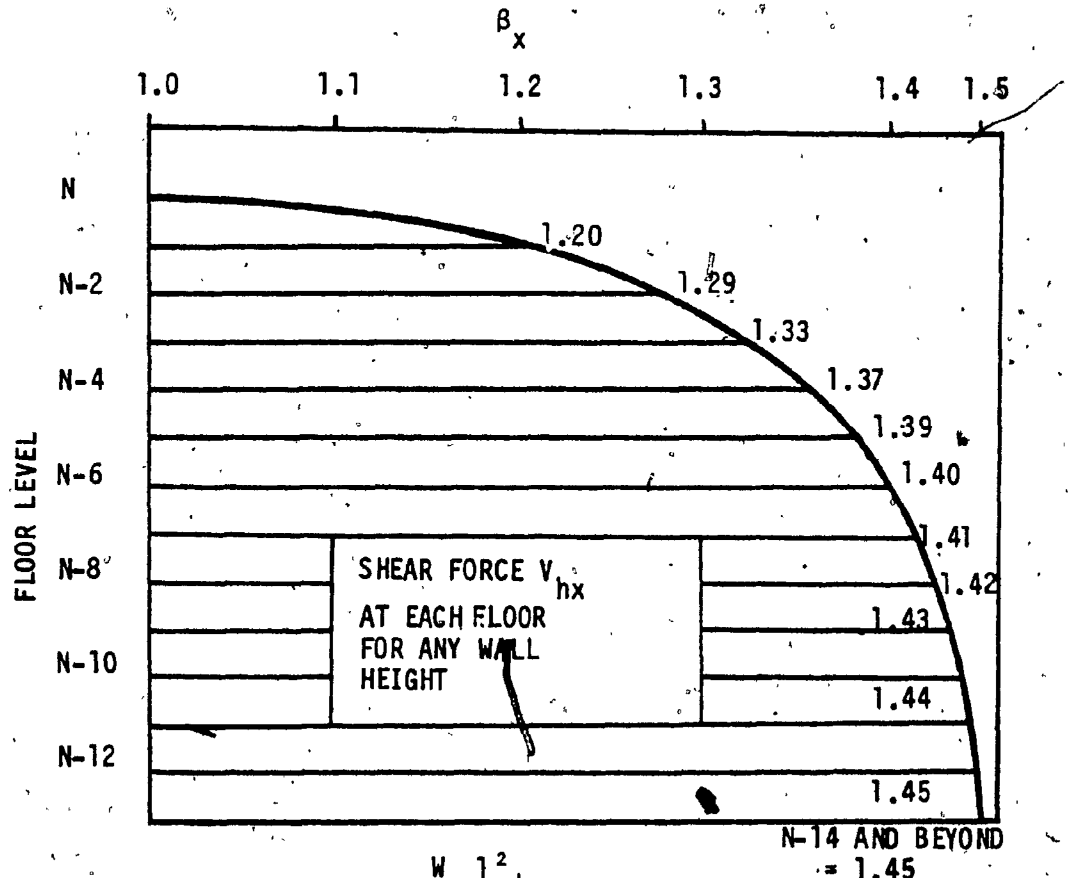


FIGURE (18) - LOCATION OF TRANSVERSE TIES, (REF. 2)



$$V_{hx} = \beta_x \frac{W_s \, l^2_d}{2 \, h_s}$$

FIGURE (19) - DESIGN CHART FOR HORIZONTAL SHEAR FORCE AT ANY FLOOR LEVEL, (REF.2)

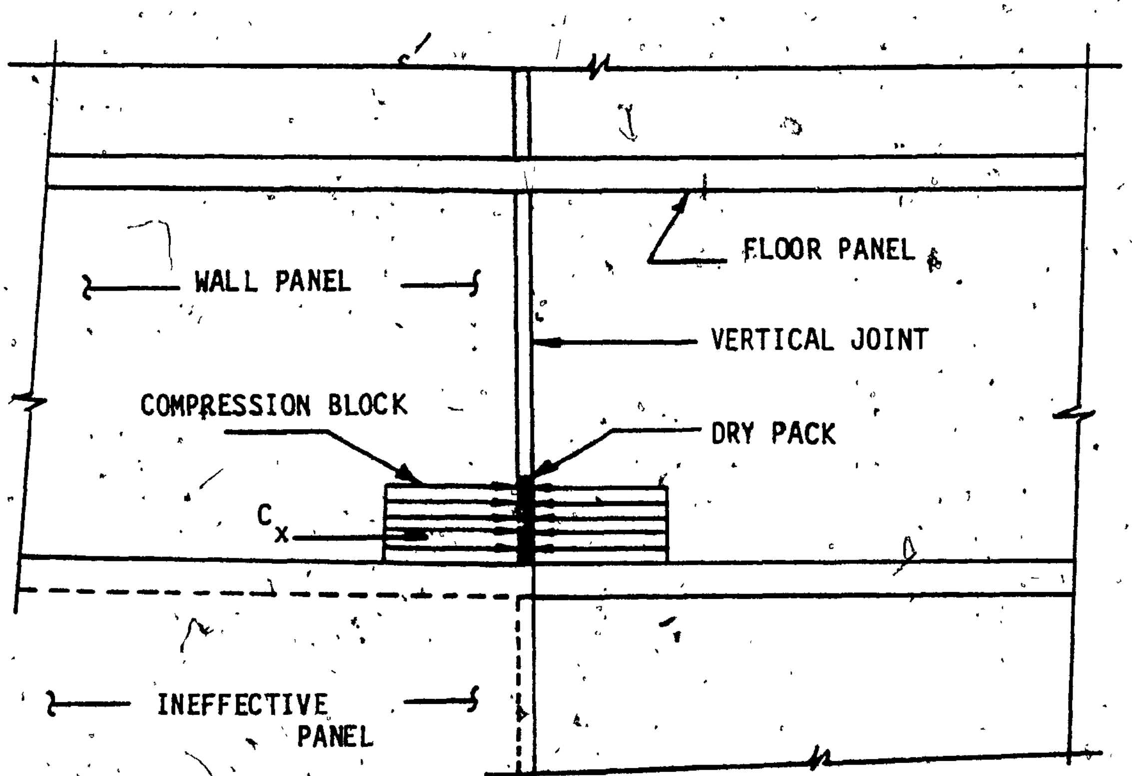


FIGURE (20) - LOCATION OF COMPRESSION BLOCK IN CANTILEVER MECHANISM, (REF.2)

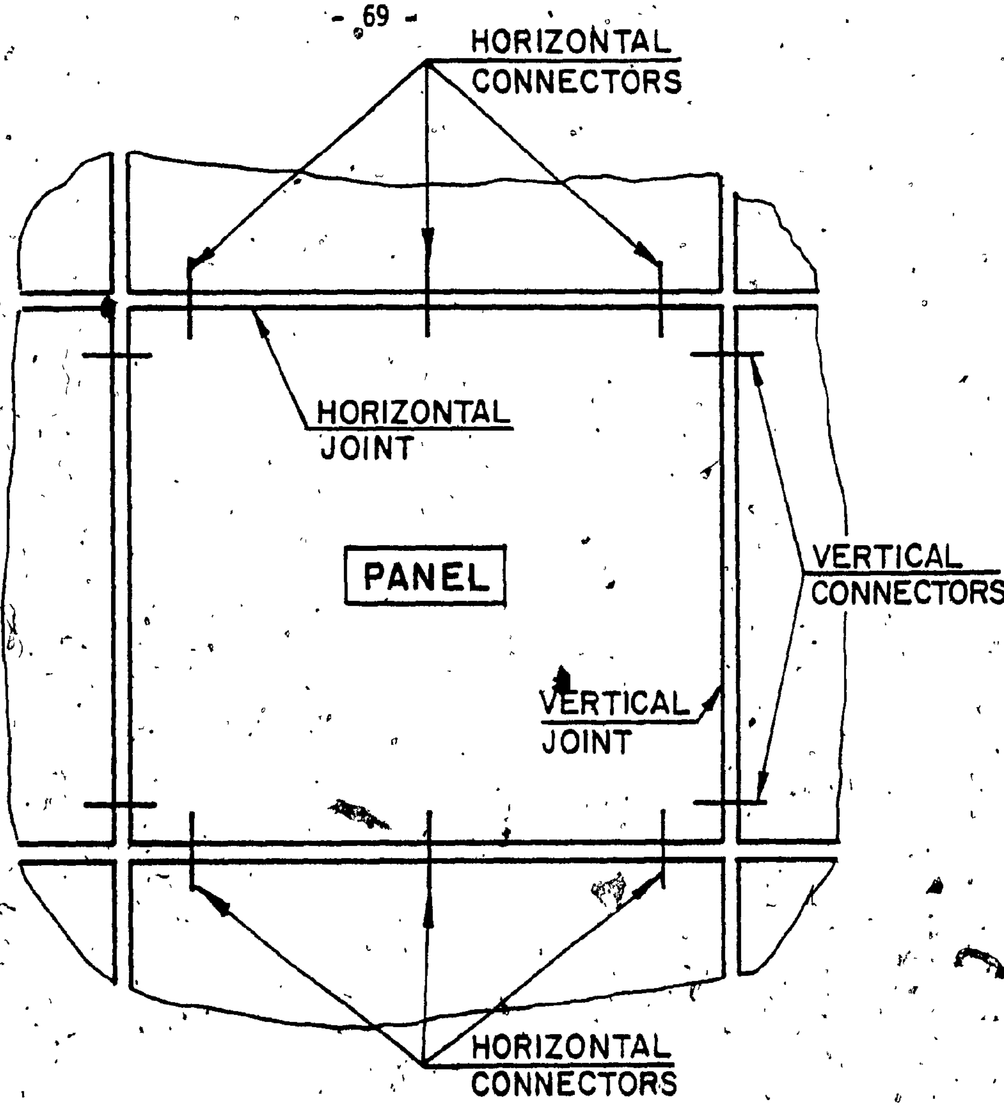


FIGURE (21) - HORIZONTAL AND VERTICAL CONNECTOR ARRANGEMENT

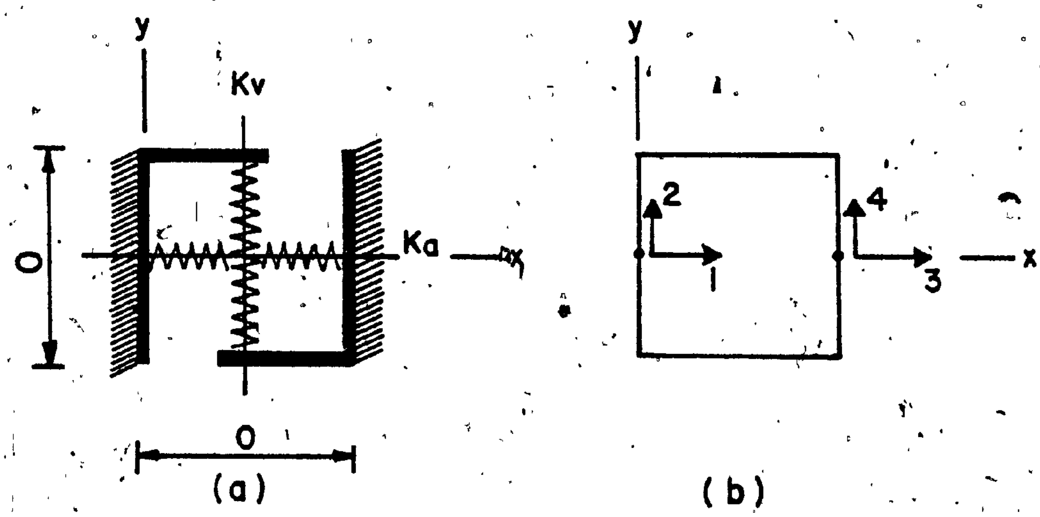
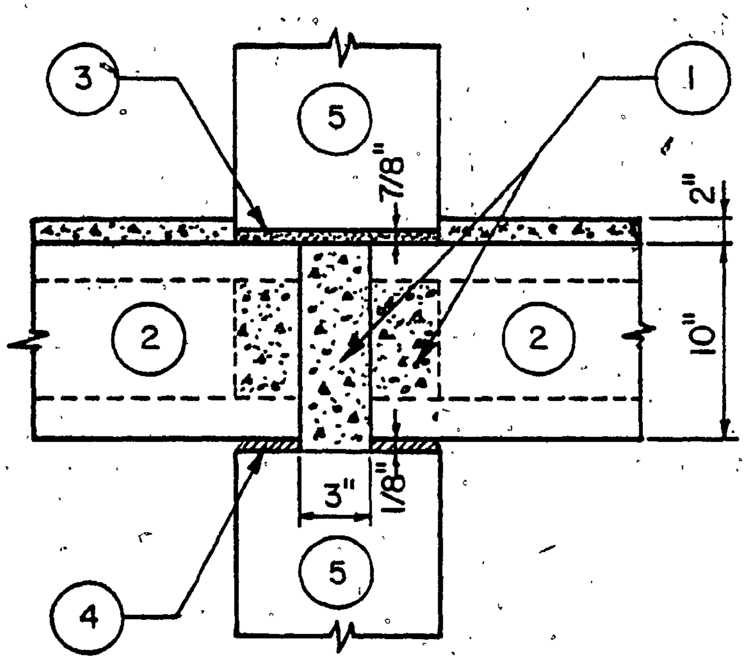


FIGURE (22) - CONNECTOR ELEMENT, (REF.7)
(a) IDEALIZATION, AND
(b) DEGREES OF FREEDOM



(1) - GROUT
(2) - FLOOR PLANK
(3) - DRY PACK
(4) - NEOPRENE BEARING PAD
(5) - PRECAST WALL PANELS

FIGURE (23) - HORIZONTAL JOINT ARRANGEMENT, (REF.4)

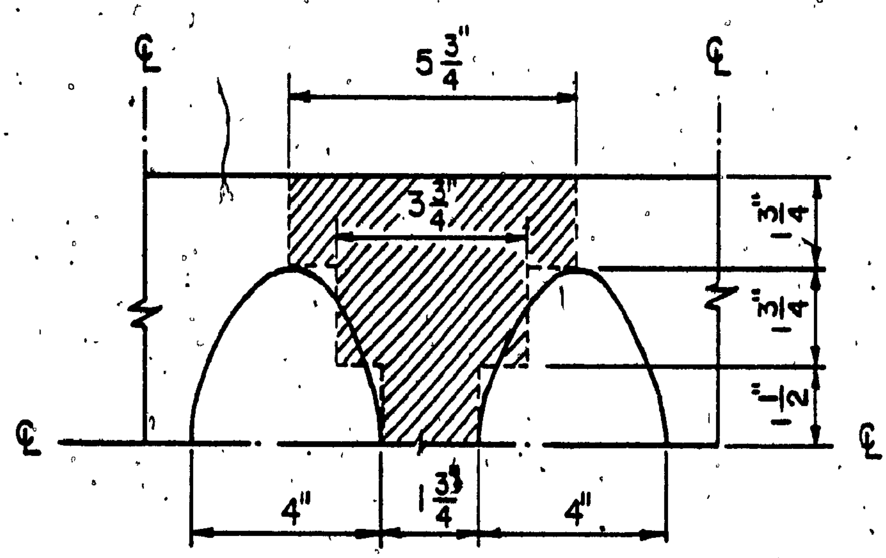


FIGURE (24) - MODEL OF HORIZONTAL CONNECTOR USED TO EVALUATE EFFECTIVE MODULUS OF ELASTICITY, (REF. 4)

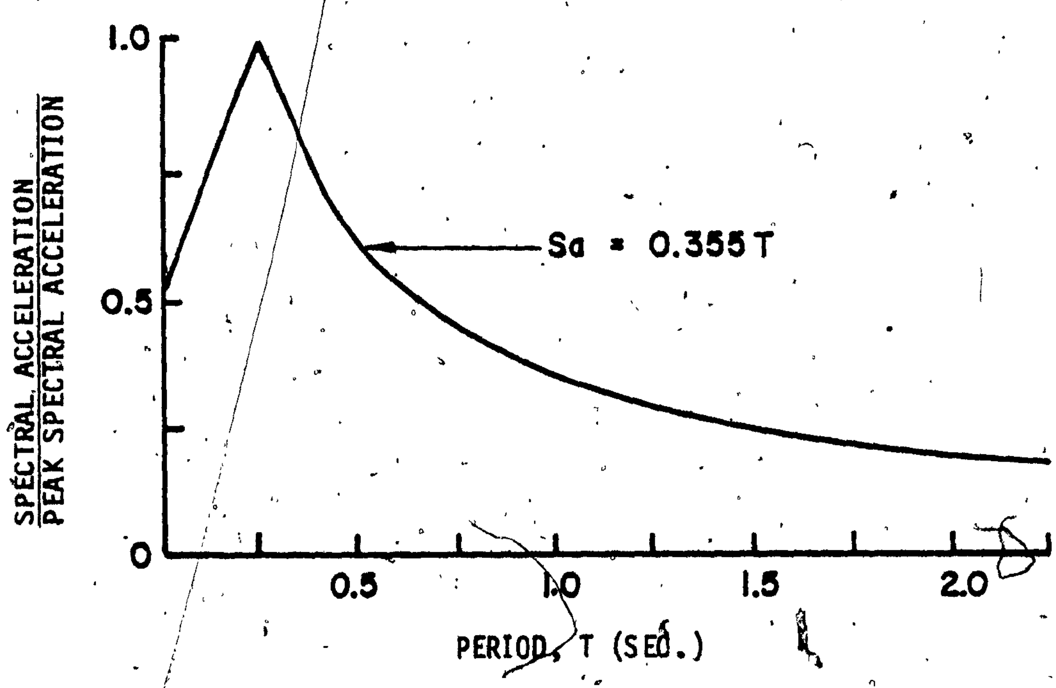


FIGURE (25) - ACCELERATION RESPONSE SPECTRA FOR 5 PER CENT DAMPING FOR EL CENTRO (1940-N.S COMPONENT), (REF.7)

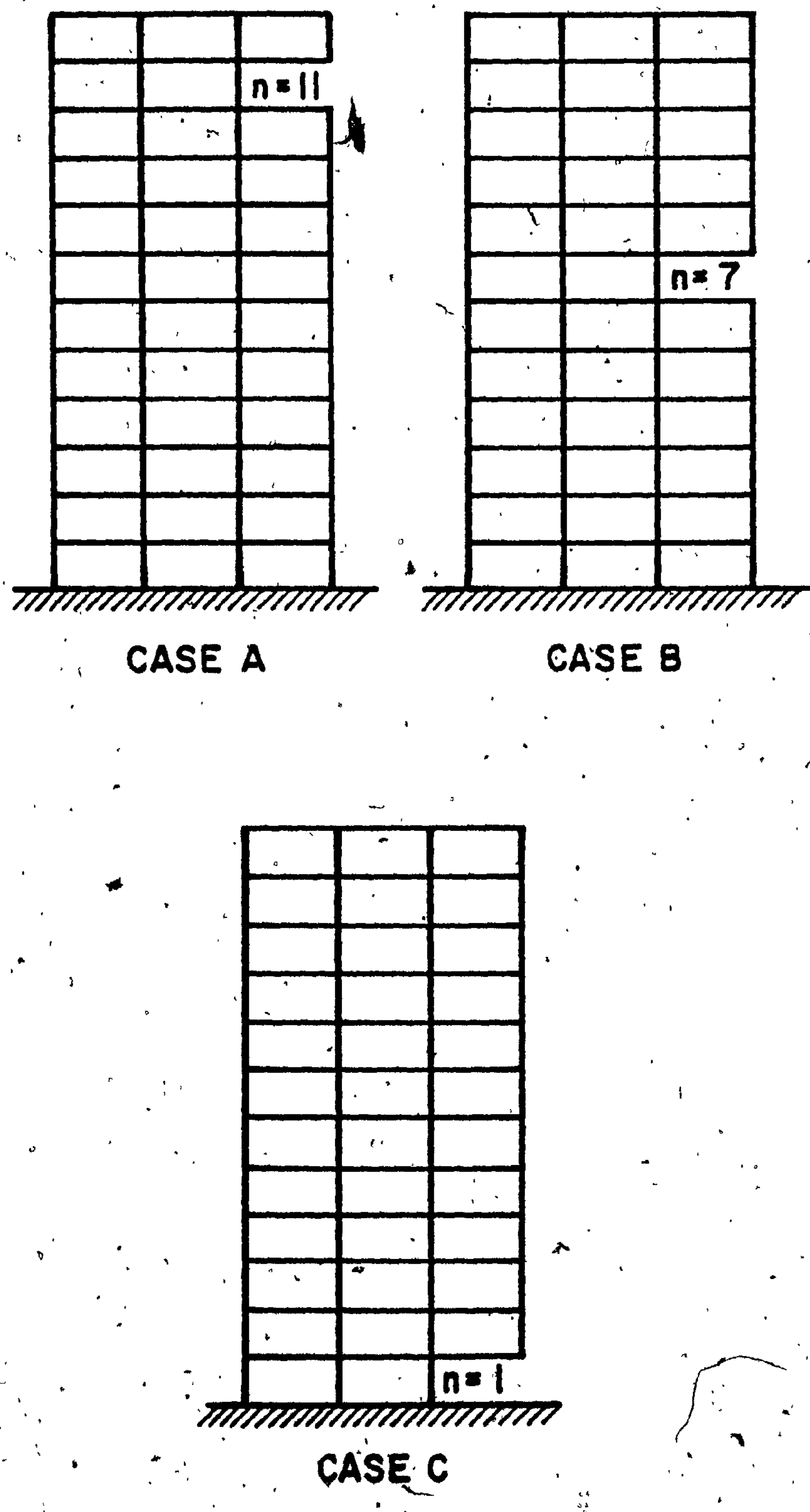


FIGURE (26) - STRUCTURE CASES STUDIED

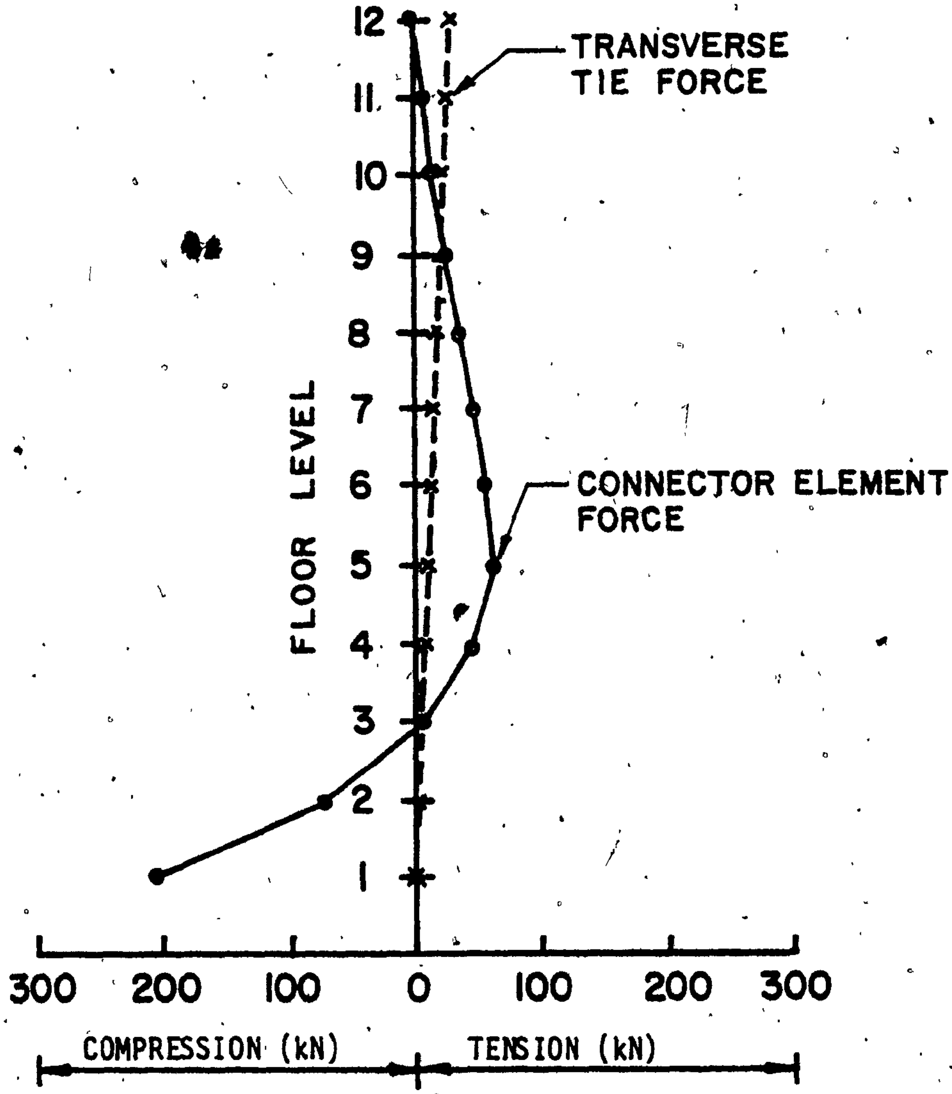


FIGURE (27 - AXIAL FORCE DISTRIBUTION FOR TRANSVERSE TIES AND VERTICAL CONNECTOR ELEMENTS FOR CANTILEVER CASE C(FIG.26), (STATIC DEAD LOAD)

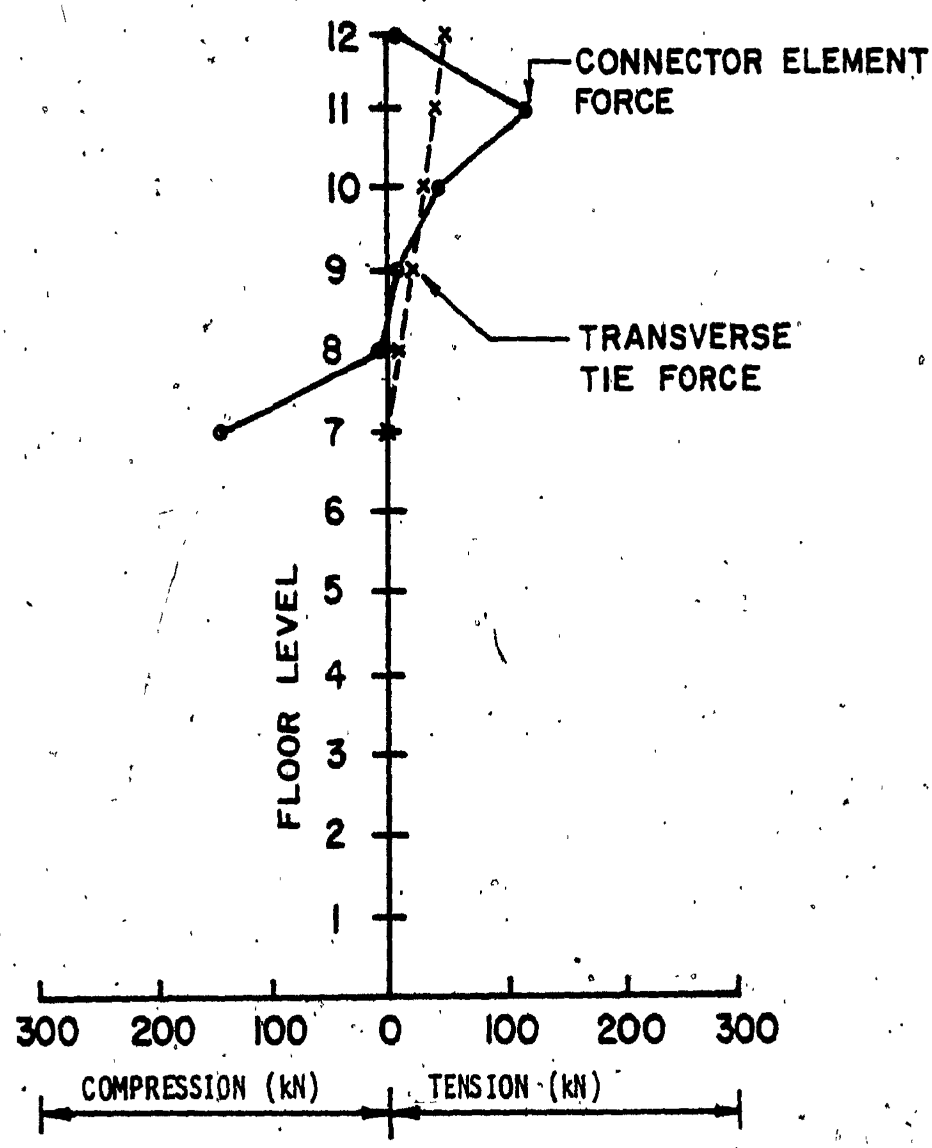


FIGURE (28) - AXIAL FORCE DISTRIBUTION FOR TRANSVERSE TIES AND VERTICAL CONNECTOR ELEMENTS FOR CANTILEVER CASE B (FIG.26), (STATIC DEAD LOAD)

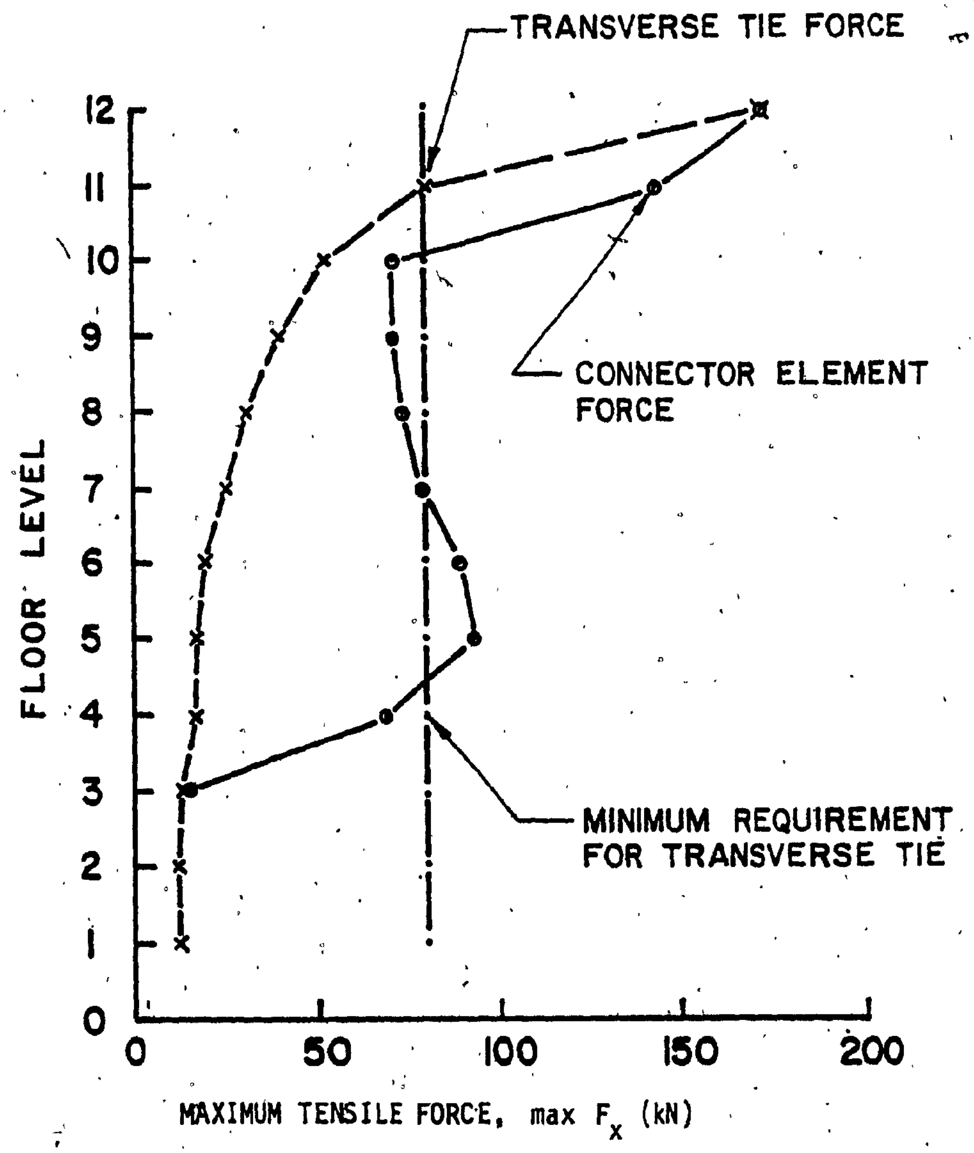


FIGURE (29) - MAXIMUM TRANSVERSE TIE FORCE AND MAXIMUM TENSILE FORCE IN VERTICAL CONNECTOR AT ANY LEVEL, (n<X<N) FOR ANY CANTILEVER DEPTH. (STATIC ANALYSIS, DEAD AND LIVE LOAD)

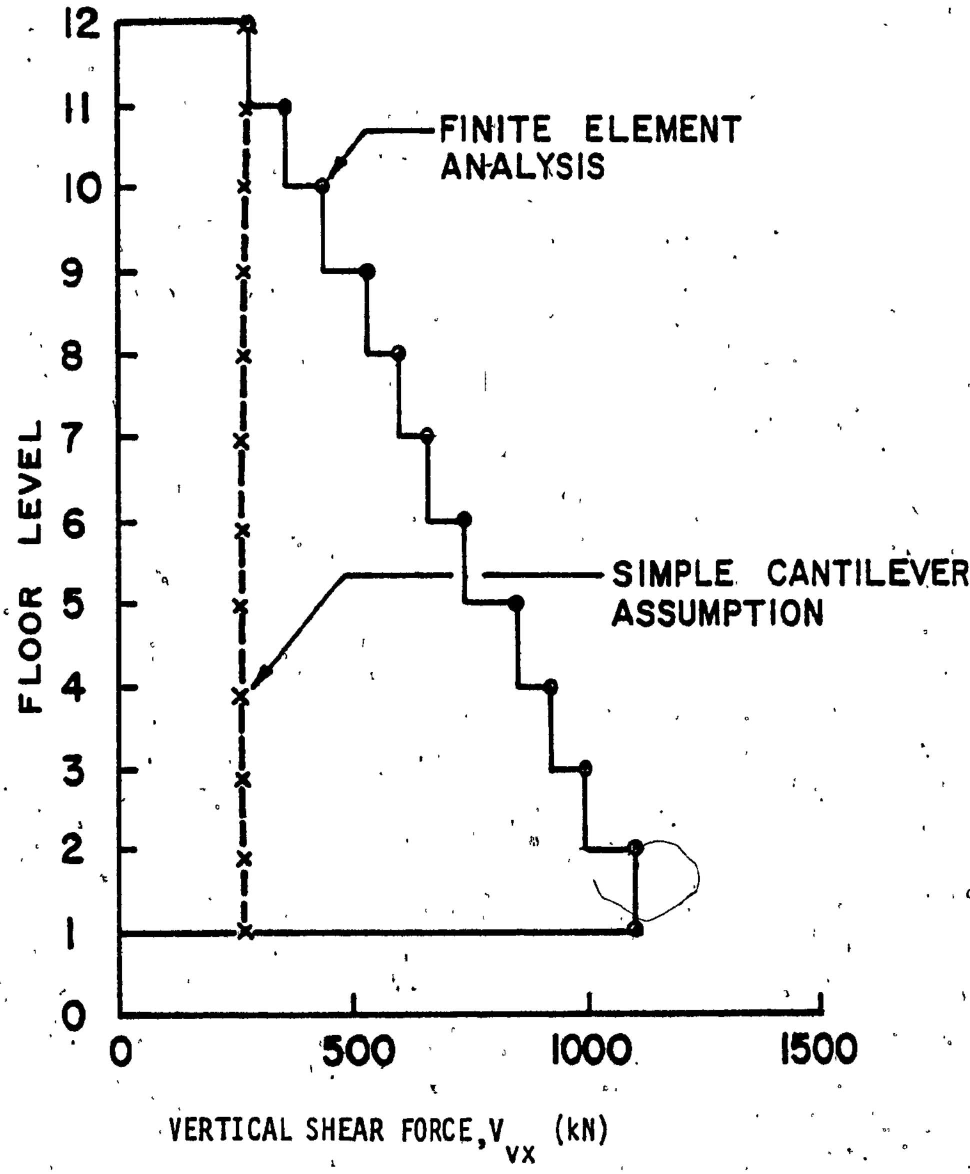
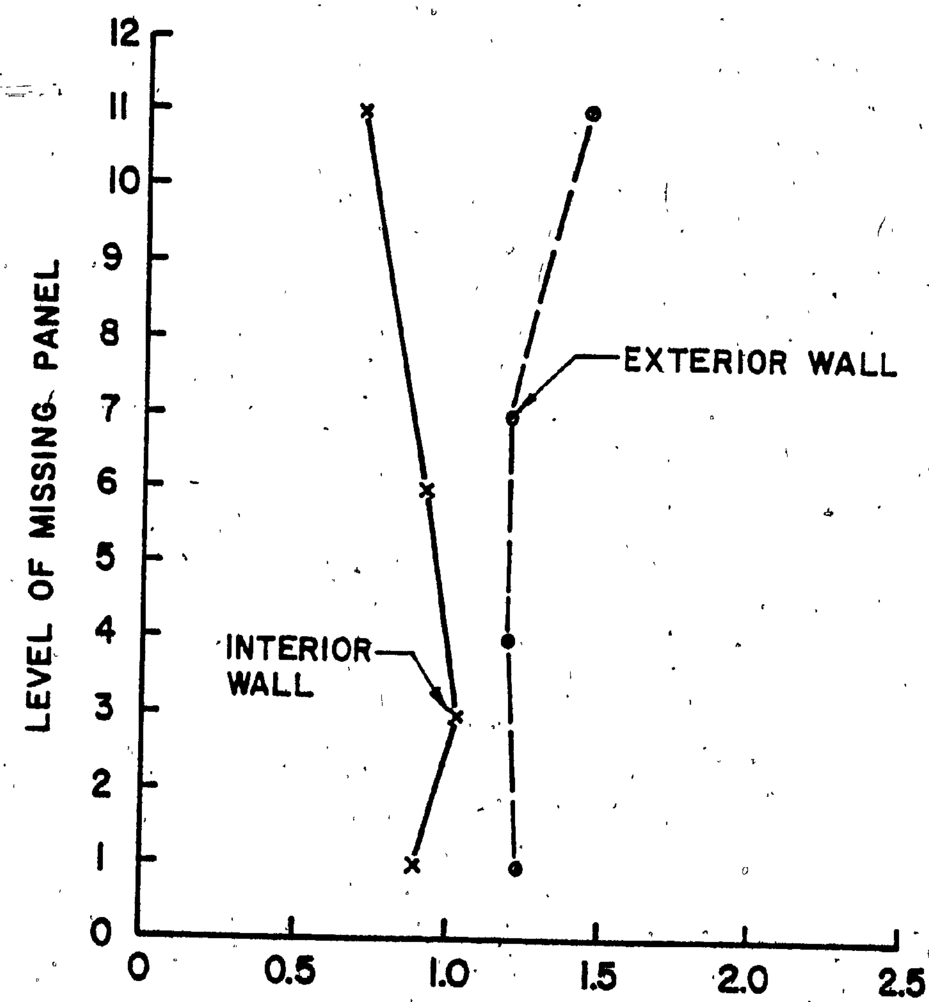


FIGURE (30) - MAXIMUM VERTICAL SHEAR FORCE FOR EACH PANEL FOR ANY CANTILEVER HEIGHT (STATIC ANALYSIS, DEAD AND LIVE LOAD)



FORCE RATIO, $F_{va}/\overline{F}_{va}$

FIGURE (31) - EFFECT OF MISSING PANEL ON AXIAL FORCE OF VERTICAL CONNECTOR (DYNAMIC ANALYSIS, $\overline{F}_{va}$ = 132 kN)

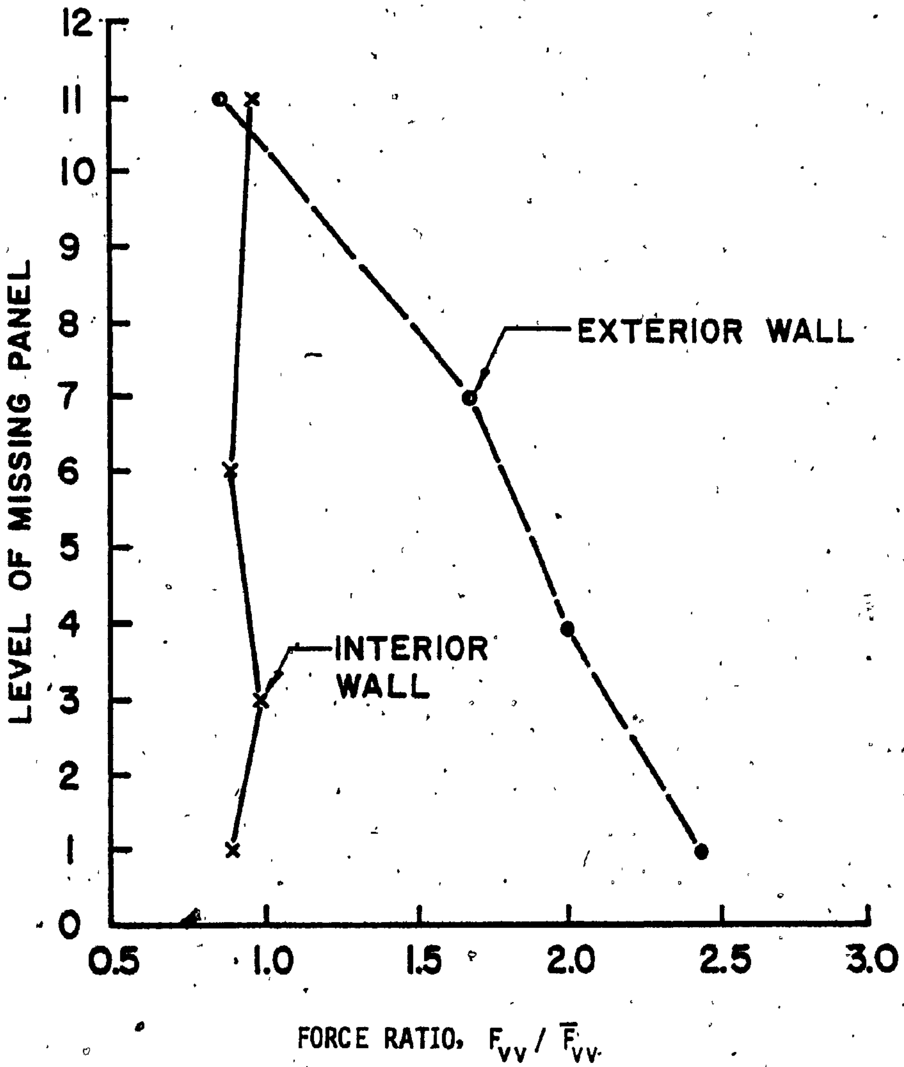


FIGURE (32) - EFFECT OF MISSING PANEL ON SHEAR FORCE OF VERTICAL CONNECTOR (DYNAMIC ANALYSIS, $\overline{F}_{VV}$ = 310 kN)

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[1] Lewicki, B., "Buildings With Large Prefabricates". Elsevier Publication Company, London, 1966.

[2] Schultz, Donald M., Burnett, Eric F.P., Fintel, Mark, "Design and Construction of Large-Panel Concrete Structure, Contract No. H-2131R, Report No. 4., A Design Approach To General Structural Integrity". Portland Cement Association, October 1977.

[3] Becker, J.M.,Llorente, C., Mueller, P.,"Seismic Response Of Large-Panel Precast Concrete Shear-Walls", ASCE Convention And Exposition, 1979.

[4] Frank, A.R., "Dynamic Modeling Of Large Precast Panel Buildings Using Finite Element With Substructuring". Report No. R76-36, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge Massachusetts, 1976.

[5] Zeck, U.I., "Joints In Large-Panel Precast Concrete Structures", Report No. R76-16, Department of Civil Engineering, Massachusetts Institute Of Technology, Cambridge, Massachusetts, January 1976.

[6] Pall, A.S., "Limited Slip Bolted Joints", A Device To Control The Seismic Response Of Large Panel Structures", Centre For Building Studies, Concordia University, Montreal, Quebec, 1979.

[7] Huttelmaier, H.P., "Dynamic Analysis Of Shear Walls And Discretely Connected Panel Structures Using A Super Finite Element Technique", Phd. Thesis, Concordia University, Montreal, Quebec 1978.

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[9] "Commentaries On Part 4 Of The National Building Code Of Canada 1977, Supplement No. 4 To The National Building Code Of Canada", Associate Committee On the National Building Code, National Research Council Of Canada, Ottawa.

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[14] Bathe, K.J., Wilson, E.L., Peterson, F.E., "SAP IV, A Structural Analysis Program For Static And Dynamic Response Of Linear Systems", Report No. EERC 73-11, College Of Engineering, University Of California, Berkeley, California, June 1973, Revised April 1974.

APPENDIX A

A-1

DATA FOR FIGURE (27)

AXIAL FORCE DISTRIBUTION FOR TRANSVERSE

TIES AND VERTICAL CONNECTOR ELEMENTS

FOR CANTILEVER CASE C (FIG.26)

(STATIC DEAD LOAD)

FLOOR LEVEL	AXIAL FORCE, (kN)		
	VERTICAL CONNECTOR		TRANSVERSE TIE,
	TENSION	COMPRESSION	TENSION
12	0.0		39.2
11	3.1		35.7
10	16.8		32.2
9	31.5		28.6
8	47.2		25.1
7	63.0		21.6
6	75.3		17.6
5	77.8		14.1
4	59.0		10.6
3	7.1		7.0
2		98.7	3.0
1		277.0	0.0

A-2

DATA FOR FIGURE (28)

AXIAL FORCE DISTRIBUTION FOR TRANSVERSE TIES AND VERTICAL CONNECTOR ELEMENTS FOR CANTILEVER CASE B (FIG.26)

(STATIC DEAD LOAD)

FLOOR LEVEL	AXIAL FORCE (kN)		
	VERTICAL CONNECTOR		TRANSVERSE TIE TENSION
	TENSION	COMPRESSION	
12	7.3		80.3
11	106.0		64.3
10	63.7		48.2
9	10.0		32.1
8		6.8	16.0
7		185.0	0

A-3

DATA FOR FIGURE (29)

MAXIMUM TRANSVERSE TIE FORCE AND MAXIMUM

TENSILE FORCE IN VERTICAL CONNECTOR

AT ANY LEVEL

(STATIC, DEAD & LIVE LOAD)

FLOOR LEVEL	MAX. TENSILE FORCE (kN)	
	TRANSVERSE TIE	VERTICAL CONNECTOR
12	178.3	178.2
11	80.3	142.1
10	52.0	73.3
9	37.4	71.0
8	30.3	72.0
7	25.0	78.0
6	21.4	89.0
5	17.8	89.4
4	17.8	69.0
3	17.8	8.0
2	17.8	0
1	17.8	0
MINIMUM REQUIREMENT = 80 kN		

A-4

DATA FOR FIGURE (30)

VERTICAL SHEAR FORCE FOR

EACH PANEL

(STATIC ANALYSIS, DEAD & LIVE LOAD)

FLOOR LEVEL	VERTICAL SHEAR FORCE, (kN)	
	FINITE ELEMENT ANALYSIS	SIMPLE CANTILEVER ASSUMPTION
12	288.7	288.7
11	360.0	288.7
10	450.0	288.7
9	540.0	288.7
8	600.0	288.7
7	670.0	288.7
6	750.0	288.7
5	860.0	288.7
4	730.0	288.7
3	1000.0	288.7
2	1100.0	288.7

A-5

DATA FOR FIGURES (31) AND (32)

EFFECT OF MISSING PANEL ON

VERTICAL CONNECTOR FORCES

(DYNAMIC LOADING)

LEVEL OF MISSING PANEL	VERTICAL CONNECTORS: RATIO OF AXIAL FORCE $F_{va}/\overline{F}_{va}$ FOR FIG. 31		RATIO OF SHEAR FORCE $F_{vv}/\overline{F}_{vv}$ FOR FIG. 32	
	INTERIOR WALL	EXTERIOR WALL	INTERIOR WALL	EXTERIOR WALL
11	0.68	1.45	0.94	0.87
7	----	1.20	----	1.67
6	0.93	----	0.89	----
4	----	1.20	----	2.20
3	1.00	----	1.00	----
1	0.90	1.34	0.91	2.41
	$\overline{F}_{va}$ = 132 kN		$\overline{F}_{vv}$ = 310 kN	

A-6

DATA FOR FIGURES (33) AND (34)

EFFECT OF MISSING PANEL ON

HORIZONTAL CONNECTOR FORCES

(DYNAMIC LOAD)

LEVEL OF MISSING PANEL	HORIZONTAL CONNECTORS			
	RATIO OF AXIAL FORCE $F_{ha}/\overline{F}_{ha}$ (FOR FIG. 33)		RATIO OF SHEAR FORCE $F_{hv}/\overline{F}_{hv}$ FOR FIG. (34)	
	INTERIOR WALL	EXTERIOR WALL	INTERIOR WALL	EXTERIOR WALL
11	0.92	0.92	0.94	0.92
7	----	0.78	----	0.85
6	0.88	----	0.84	----
4	----	0.91	----	0.93
3	0.99	----	1.13	----
1	0.99	1.24	1.28	1.44
	$\overline{F}_{ha}$ = 2501 kN		$\overline{F}_{hv}$ = 535 kN	

A-7

DATA FOR FIGURES (35) AND (36)

EFFECT OF MISSING PANEL ON

FUNDAMENTAL PERIOD AND MAXIMUM DEFLECTION

(DYNAMIC LOADING)

LEVEL OF MISSING PANEL	RATIO OF FUNDAMENTAL PERIOD $T_s/\overline{T}_s$ (FOR FIG.35)		RATIO OF MAXIMUM DEFLECTION $\Delta_s/\overline{\Delta}_s$ (FOR FIG.36)	
	INTERIOR WALL	EXTERIOR WALL	INTERIOR WALL	EXTERIOR WALL
11	1.00	0.97	1.02	0.95
7	----	1.14	----	1.12
6	1.08	----	1.07	----
4	----	1.42	----	1.47
3	1.08	----	1.07	----
1	1.01	1.55	1.02	1.68
	$\overline{T}_s$ = 0.755 secs.		$\overline{\Delta}_s$ = 60 mm.	

A-8

DATA FOR FIGURES (37) AND (38)

EFFECT OF MISSING PANEL ON BASE

OVERTURNING MOMENT AND BASE SHEAR

(DYNAMIC ANALYSIS)

LEVEL OF MISSING PANEL	RATIO OF BASE OVERTURNING MOMENT, $M_o/\bar{M}_o$ (FOR FIG.37)		RATIO OF BASE SHEAR $V_s/\bar{V}_s$ (FOR FIG. 38)	
	INTERIOR WALL	EXTERIOR WALL	INTERIOR WALL	EXTERIOR WALL
11	0.88	0.90	0.90	0.82
7	----	0.77	----	0.75
6	0.83	----	0.83	----
4	----	0.63	----	0.70
3	0.86	----	0.90	----
1	0.90	0.73	1.03	0.86
	$\bar{M}_o$ = 4.44x10⁴ kn-m		$\bar{V}_s$ = 1951 kN	

APPENDIX B

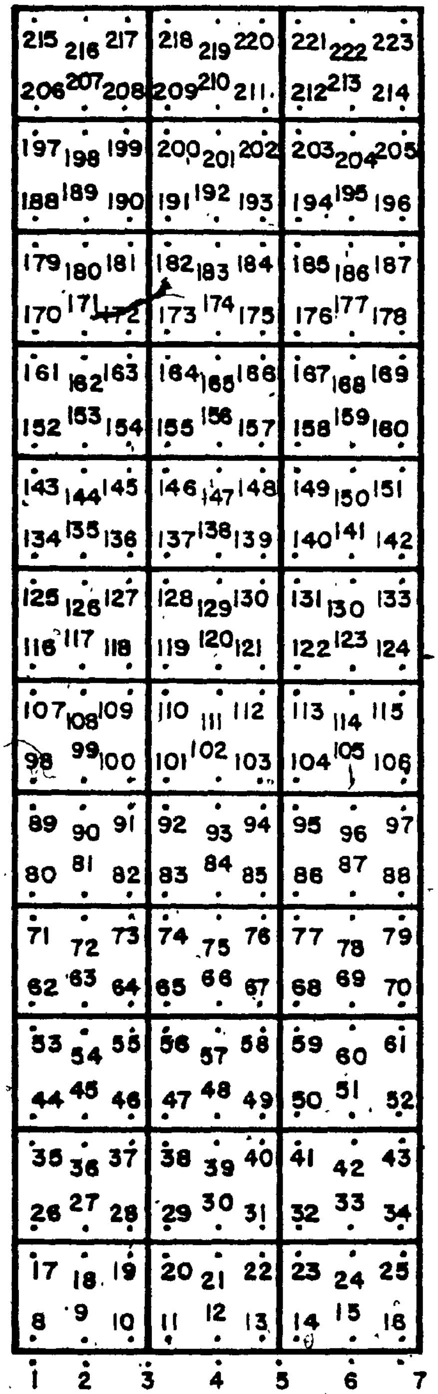
B-1

12	24	36
11	23	35
10	22	34
9	21	33
8	20	32
7	19	31
6	18	30
5	17	29
4	16	28
3	15	27
2	14	26
1	13	25

PANEL NUMBERING

(STRUCTURAL MODEL FOR FINITE ELEMENT ANALYSIS)

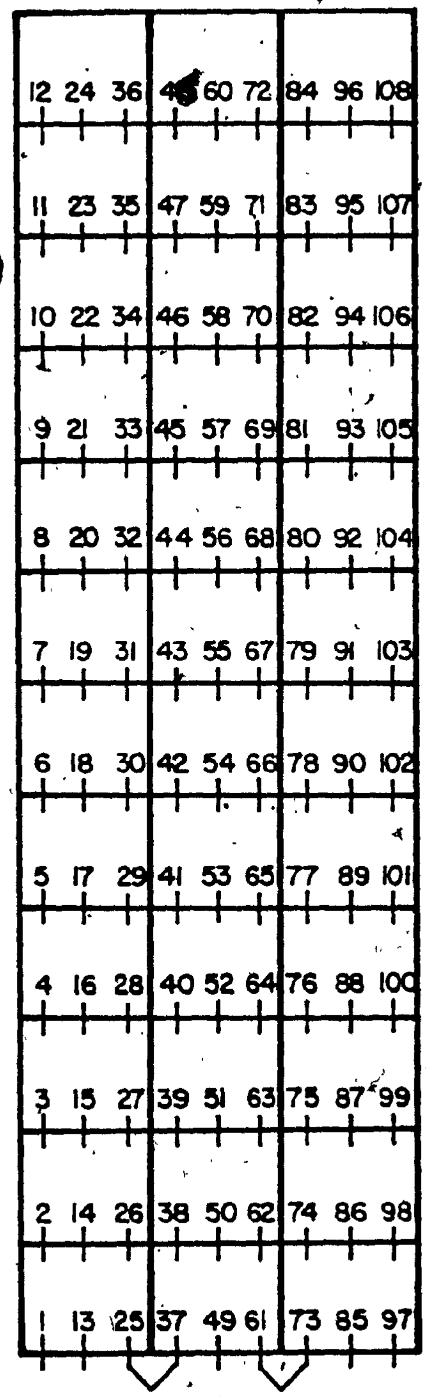
B-2.



NODAL NUMBERING

(STRUCTURAL MODEL FOR FINITE ELEMENT ANALYSIS)

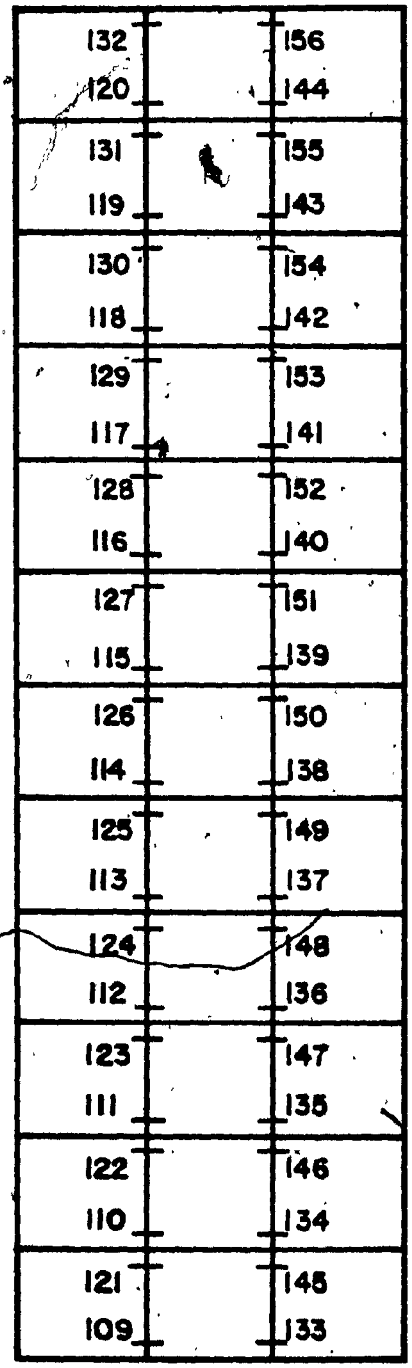
B-3



HORIZONTAL CONNECTOR NUMBERING

(STRUCTURAL MODEL FOR FINITE ELEMENT ANALYSIS)

B-4



VERTICAL CONNECTOR NUMBERING

(STRUCTURAL MODEL FOR FINITE ELEMENT ANALYSIS)

APPENDIX C

C-1

INPUT DATA FOR STATIC LOADING

(FINITE ELEMENT ANALYSIS)

```
12 STOREY  SHEAR-WALL.  STATIC  LOADING.  FINITE ELEMENT ANALYSIS.
 223   2   1   0   0   0   0   0
   1   1   1   1   1   1   1        0        0        0   0        0
   2   1   1   1   1   1   1        0        0        0   0        0
   3   1   1   1   1   1   1        0        0        0   0        0
   4   1   1   1   1   1   1        0        0        0   0        0
   5   1   1   1   1   1   1        0        0        0   0        0
   6   1   1   1   1   1   1        0        0        0   0        0
   7   1   1   1   1   1   1        0        0        0   0        0
   8   0   0   1   1   1   1        0        0        0   0        0
 206   0   0   1   1   1   1        0        0        0  18        0
  17   0   0   1   1   1   1        0        0        0   0        0
 215   0   0   1   1   1   1        0        0        0  18        0
   9   0   0   1   1   1   1        0        0        0   0        0
 207   0   0   1   1   1   1        0        0        0  18        0
  18   0   0   1   1   1   1        0        0        0   0        0
 216   0   0   1   1   1   1        0        0        0  18        0
  10   0   0   1   1   1   1        0        0        0   0        0
 208   0   0   1   1   1   1        0        0        0  18        0
  19   0   0   1   1   1   1        0        0        0   0        0
 217   0   0   1   1   1   1        0        0        0  18        0
  11   0   0   1   1   1   1        0        0        0   0        0
 209   0   0   1   1   1   1        0        0        0  18        0
  20   0   0   1   1   1   1        0        0        0   0        0
 218   0   0   1   1   1   1        0        0        0  18        0
  12   0   0   1   1   1   1        0        0        0   0        0
 210   0   0   1   1   1   1        0        0        0  18        0
  21   0   0   1   1   1   1        0        0        0   0        0
 219   0   0   1   1   1   1        0        0        0  18        0
  13   0   0   1   1   1   1        0        0        0   0        0
 211   0   0   1   1   1   1        0        0        0  18        0
  22   0   0   1   1   1   1        0        0        0   0        0
 220   0   0   1   1   1   1        0        0        0  18        0
  14   0   0   1   1   1   1        0        0        0   0        0
 212   0   0   1   1   1   1        0        0        0  18        0
  23   0   0   1   1   1   1        0        0        0   0        0
 221   0   0   1   1   1   1        0        0        0  18        0
  15   0   0   1   1   1   1        0        0        0   0        0
 213   0   0   1   1   1   1        0        0        0  18        0
  24   0   0   1   1   1   1        0        0        0   0        0
 222   0   0   1   1   1   1        0        0        0  18        0
  16   0   0   1   1   1   1        0        0        0   0        0
 214   0   0   1   1   1   1        0        0        0  18        0
  25   0   0   1   1   1   1        0        0        0   0        0
 223   0   0   1   1   1   1        0        0        0  18        0
   9  36   2   2   2   0
   1     27.6      .17       200 .000000000024
   2    .0003   .00017        .2 .000000000002
   1   6   8   9  10  19  18  17   0
0  1   5   2   1    3670     2970     74[illegible].5     0   0
   3 100
   1

   2   6  26  27  28  37  36  [illegible]  0
```

1	3	6	44	45	46	55	54	53	0
1	4	6	62	63	64	73	72	71	0
1	5	6	80	81	82	91	90	89	0
1	6	6	98	99	100	109	108	107	0
1	7	6	116	117	118	127	126	125	0
1	8	6	134	135	136	145	144	143	0
1	9	6	152	153	154	163	162	161	0
1	10	6	170	171	172	181	180	179	0
1	11	6	188	189	190	199	198	197	0
1	12	6	206	207	208	217	216	215	0
1	13	6	11	12	13	22	21	20	0
1	14	6	29	30	31	40	39	38	0
1	15	6	47	48	49	58	57	56	0
1	16	6	65	66	67	76	75	74	0
1	17	6	83	84	85	94	93	92	0
1	18	6	101	102	103	112	111	110	0
1	19	6	119	120	121	130	129	128	0
1	20	6	137	138	139	148	147	146	0
1	21	6	155	156	157	166	165	164	0
1	22	6	173	174	175	184	183	182	0
1	23	6	191	192	193	202	201	200	0
1	24	6	209	210	211	220	219	218	0
1	25	6	14	15	16	25	24	23	0
1	26	6	32	33	34	43	42	41	0
1	27	6	50	51	52	61	60	59	0
1	28	6	68	69	70	79	78	77	0
1	29	6	86	87	88	97	96	95	0
1	30	6	104	105	106	115	114	113	0
1	31	6	122	123	124	133	132	131	0
1	32	6	140	141	142	151	150	149	0
1	33	6	158	159	160	169	168	167	0
1	34	6	176	177	178	187	186	185	0
1	[illegible]	[illegible]	194	195	196	205	204	203	0

1

36	6	212	213	214	223	222	221	0		
1										
10	156	3								
1		14.6		640						
2		8000		5300						
3		16000		10600						
1	1	8	2	2	2	0				
13	2	9	3	2	2	0				
25	3	10	2	2	2	0				
37	3	11	2	2	2	0				
49	4	12	3	2	2	0				
61	5	13	2	2	2	0				
73	5	14	2	2	2	0				
85	6	15	3	2	2	0				
97	7	16	2	2	2	0				
2	17	26	2	2	2	1				
12	197	206								
14	18	27	3	2	2	1				
24	198	207								
26	19	28	2	2	2	1				
36	199	208								
38	20	29	2	2	2	1				
48	200	209								
50	21	30	3	2	2	1				
60	201	210								
62	22	31	2	2	2	1				
72	202	211								
74	23	32	2	2	2	1				
84	203	212								
86	24	33	3	2	2	1				
96	204	213								
98	25	34	2	2	2	1				
108	205	214								
109	10	11	1	1	2	1				
120	208	209								
121	19	20	1	[illegible]	2	1				
132	217	218								
133	13	14	1	1	2	1				
144	211	212								
145	22	23	1	1	2	1				
156	220	221								
17	1		0	63.			0	0	0	0
18	1		0	126.			0	0	0	0
19	1		0	63.			0	0	0	0
20	1		0	63.			0	0	0	0
21	1		0	126.			0	0	0	0
22	1		0	63.			0	0	0	0
23	1		0	63.			0	0	0	0
24	1		0	126.			0	0	0	0
25	1		0	63.			0	0	0	0
35	1		0	63.			0	0	0	0
36	1		0	126.			0	0	0	0
37	1		0	63.			0	0	0	0
38	1		0	63.			0	0	0	0
39	1		0	126.			0	0	0	0
40	1		0	63.			0	0	0	0
41	1		0	63.			0	0	0	0
42	1		0	126.			0	0	0	0
43	1		0	63.			0	0	0	0
53	1		0	63.			0	0	0	0
54	1		0	126.			0	0	0	0
55	1		0	63.			0	0	0	0
56	1		0	63.			0	0	0	0
57	1		0	126.			0	0	0	0

59	1	0	63.	0	0	0	0
60	1	0	126.	0	0	0	0
61	1	0	63.	0	0	0	0
71	1	0	63.	0	0	0	0
72	1	0	126.	0	0	0	0
73	1	0	63.	0	0	0	0
74	1	0	63.	0	0	0	0
75	1	0	126.	0	0	0	0
76	1	0	63.	0	0	0	0
77	1	0	63.	0	0	0	0
78	1	0	126.	0	0	0	0
79	1	0	63.	0	0	0	0
89	1	0	63.	0	0	0	0
90	1	0	126.	0	0	0	0
91	1	0	63.	0	0	0	0
92	1	0	63.	0	0	0	0
93	1	0	126.	0	0	0	0
94	1	0	63.	0	0	0	0
95	1	0	63.	0	0	0	0
96	1	0	126.	0	0	0	0
97	1	0	63.	0	0	0	0
107	1	0	63.	0	0	0	0
108	1	0	126.	0	0	0	0
109	1	0	63.	0	0	0	0
110	1	0	63.	0	0	0	0
111	1	0	126.	0	0	0	0
112	1	0	63.	0	0	0	0
113	1	0	63.	0	0	0	0
114	1	0	126.	0	0	0	0
115	1	0	63.	0	0	0	0
125	1	0	63.	0	0	0	0
126	1	0	126.	0	0	0	0
127	1	0	63.	0	0	0	0
128	1	0	63.	0	0	0	0
129	1	0	126.	0	0	0	0
130	1	0	63.	0	0	0	0
131	1	0	63.	0	0	0	0
132	1	0	126.	0	0	0	0
133	1	0	63.	0	0	0	0
143	1	0	63.	0	0	0	0
144	1	0	126.	0	0	0	0
145	1	0	63.	0	0	0	0
146	1	0	63.	0	0	0	0
147	1	0	126.	0	0	0	0
148	1	0	63.	0	0	0	0
149	1	0	63.	0	0	0	0
150	1	0	126.	0	0	0	0
151	1	0	63.	0	0	0	0
161	1	0	63.	0	0	0	0
162	1	0	126.	0	0	0	0
163	1	0	63.	0	0	0	0
164	1	0	63.	0	0	0	0
165	1	0	126.	0	0	0	0
166	1	0	63.	0	0	0	0
167	1	0	63.	0	0	0	0
168	1	0	126.	0	0	0	0
169	1	0	63.	0	0	0	0
179	1	0	63.	0	0	0	0
180	1	0	126.	0	0	0	0
181	1	0	63.	0	0	0	0
182	1	0	63.	0	0	0	0
183	1	0	126.	0	0	0	0
184	1	0	63.	0	0	0	0
185	1	0	63.	0	0	0	0
186	1	0	126.	0	0	0	0
187	1	0	63.	0	0	0	0

197	1	0	63.	0	0	0	0
198	1	0	126.	0	0	0	0
199	1	0	63.	0	0	0	0
200	1	0	63.	0	0	0	0
201	1	0	126.	0	0	0	0
202	1	0	63.	0	0	0	0
203	1	0	63.	0	0	0	0
204	1	0	126.	0	0	0	0
205	1	0	63.	0	0	0	0
215	1	0	63.	0	0	0	0
216	1	0	126.	0	0	0	0
217	1	0	63.	0	0	0	0
218	1	0	63.	0	0	0	0
219	1	0	126.	0	0	0	0
220	1	0	63.	0	0	0	0
221	1	0	63.	0	0	0	0
222	1	0	126.	0	0	0	0
223	1	0	63.	0	0	0	0

C-2

INPUT DATA FOR DYNAMIC LOADING,

(FINITE ELEMENT ANALYSIS)

```
12 STOREY SHEAR-WALL.  DYNAMIC LOADING.  FINITE ELEMENT ANALYSIS.
 223   3   0  12   3   0   0   0
   1   1   1   1   1   1   1        0        0        0    0        0
   2   1   1   1   1   1   1        0        0        0    0        0
   3   1   1   1   1   1   1        0        0        0    0        0
   4   1   1   1   1   1   1        0        0        0    0        0
   5   1   1   1   1   1   1        0        0        0    0        0
   6   1   1   1   1   1   1        0        0        0    0        0
   7   1   1   1   1   1   1        0        0        0    0        0
   8   0   0   1   1   1   1        0        0        0    0        0
 206   0   0   1   1   1   1        0        0        0   18        0
  17   0   0   1   1   1   1        0        0        0    0        0
 215   0   0   1   1   1   1        0        0        0   18        0
   9   0   0   1   1   1   1        0        0        0    0        0
 207   0   0   1   1   1   1        0        0        0   18        0
  18   0   0   1   1   1   1        0        0        0    0        0
 216   0   0   1   1   1   1        0        0        0   18        0
  10   0   0   1   1   1   1        0        0        0    0        0
 208   0   0   1   1   1   1        0        0        0   18        0
  19   0   0   1   1   1   1        0        0        0    0        0
 217   0   0   1   1   1   1        0        0        0   18        0
  11   0   0   1   1   1   1        0        0        0    0        0
 209   0   0   1   1   1   1        0        0        0   18        0
  20   0   0   1   1   1   1        0        0        0    0        0
 218   0   0   1   1   1   1        0        0        0   18        0
  12   0   0   1   1   1   1        0        0        0    0        0
 210   0   0   1   1   1   1        0        0        0   18        0
  21   0   0   1   1   1   1        0        0        0    0        0
 219   0   0   1   1   1   1        0        0        0   18        0
  13   0   0   1   1   1   1        0        0        0    0        0
 211   0   0   1   1   1   1        0        0        0   18        0
  22   0   0   1   1   1   1        0        0        0    0        0
 220   0   0   1   1   1   1        0        0        0   18        0
  14   0   0   1   1   1   1        0        0        0    0        0
 212   0   0   1   1   1   1        0        0        0   18        0
  23   0   0   1   1   1   1        0        0        0    0        0
 221   0   0   1   1   1   1        0        0        0   18        0
  15   0   0   1   1   1   1        0        0        0    0        0
 213   0   0   1   1   1   1        0        0        0   18        0
  24   0   0   1   1   1   1        0        0        0    0        0
 222   0   0   1   1   1   1        0        0        0   18        0
  16   0   0   1   1   1   1        0        0        0    0        0
 214   0   0   1   1   1   1        0        0        0   18        0
  25   0   0   1   1   1   1        0        0        0    0        0
 223   0   0   1   1   1   1        0        0        0   18        0
   9  36   2   2   2   0
   1    27.6      .17        200 .0000000000024
   2    .0003   .00017        .2 .0000000000002
   1   6   8   9  10  19  18  17   0
0  1   5   2   1     3670     2970      742.5       0    0
   3 100
   1

   2   6  26  27  28  37  36  35   0
1
   3   6  44  45  46  55  54  53   0
1
   4   6  62  63  64  73  72  71   0
```

1	5	6	80	81	82	91	90	89	0
1	6	6	98	99	100	109	108	107	0
1	7	6	116	117	118	127	126	125	0
1	8	6	134	135	136	145	144	143	0
1	9	6	152	153	154	163	162	161	0
1	10	6	170	171	172	181	180	179	0
1	11	6	188	189	190	199	198	197	0
1	12	6	206	207	208	217	216	215	0
1	13	6	11	12	13	22	21	20	0
1	14	6	29	30	31	40	39	38	0
1	15	6	47	48	49	58	57	56	0
1	16	6	65	66	67	76	75	74	0
1	17	6	83	84	85	94	93	92	0
1	18	6	101	102	103	112	111	110	0
1	19	6	119	120	121	130	129	128	0
1	20	6	137	138	139	148	147	146	0
1	21	6	155	156	157	166	165	164	0
1	22	6	173	174	175	184	183	182	0
1	23	6	191	192	193	202	201	200	0
1	24	6	209	210	211	220	219	218	0
1	25	6	14	15	16	25	24	23	0
1	26	6	32	33	34	43	42	41	0
1	27	6	50	51	52	61	60	59	0
1	28	6	68	69	70	79	78	77	0
1	29	6	86	87	88	97	96	95	0
1	30	6	104	105	106	115	114	113	0
1	31	6	122	123	124	133	132	131	0
1	32	6	140	141	142	151	150	149	0
1	33	6	158	159	160	169	168	167	0
1	34	6	176	177	178	187	186	185	0
1	35	6	194	195	196	205	204	203	0
1	36	6	212	213	214	223	222	221	0

1		14.6		640						
2		8000		5300						
3		16000		10600						
1	1	8	2	2	2	0				
13	2	9	3	2	2	0				
25	3	10	2	2	2	0				
37	3	11	2	2	2	0				
49	4	12	3	2	2	0				
61	5	13	2	2	2	0				
73	5	14	2	2	2	0				
85	6	15	3	2	2	0				
97	7	16	2	2	2	0				
2	17	26	2	2	2	1				
12	197	206								
14	18	27	3	2	2	1				
24	198	207								
26	19	28	2	2	2	1				
36	199	208								
38	20	29	2	2	2	1				
48	200	209								
50	21	30	3	2	2	1				
60	201	210								
62	22	31	2	2	2	1				
72	202	211								
74	23	32	2	2	2	1				
84	203	212								
86	24	33	3	2	2	1				
96	204	213								
98	25	34	2	2	2	1				
108	205	214								
109	10	11	1	1	2	1				
120	208	209								
121	19	20	1	1	2	1				
132	217	218								
133	13	14	1	1	2	1				
144	211	212								
145	22	23	1	1	2	1				
156	220	221								
17	0		.005		.005		0	0	0	0
18	0		.01		.01		0	0	0	0
19	0		.005		.005		0	0	0	0
20	0		.005		.005		0	0	0	0
21	0		.01		.01		0	0	0	0
22	0		.005		.005		0	0	0	0
23	0		.005		.005		0	0	0	0
24	0		.01		.01		0	0	0	0
25	0		.005		.005		0	0	0	0
35	0		.005		.005		0	0	0	0
36	0		.01		.01		0	0	0	0
37	0		.005		.005		0	0	0	0
38	0		.005		.005		0	0	0	0
39	0		.01		.01		0	0	0	0
40	0		.005		.005		0	0	0	0
41	0		.005		.005		0	0	0	0
42	0		.01		.01		0	0	0	0
43	0		.005		.005		0	0	0	0
53	0		.005		.005		0	0	0	0
54	0		.01		.01		0	0	0	0
55	0		.005		.005		0	0	0	0
56	0		.005		.005		0	0	0	0
57	0		.01		.01		0	0	0	0
58	0		.005		.005		0	0	0	0
59	0		.005		.005		0	0	0	0
60	0		.01		.01		0	0	0	0
61	0		.005		.005		0	0	0	0
71	0		.005		.005		0	0	0	0

72	0	.01	.01	0	0	0	0
73	0	.005	.005	0	0	0	0
74	0	.005	.005	0	0	0	0
75	0	.01	.01	0	0	0	0
76	0	.005	.005	0	0	0	0
77	0	.005	.005	0	0	0	0
78	0	.01	.01	0	0	0	0
79	0	.005	.005	0	0	0	0
89	0	.005	.005	0	0	0	0
90	0	.01	.01	0	0	0	0
91	0	.005	.005	0	0	0	0
92	0	.005	.005	0	0	0	0
93	0	.01	.01	0	0	0	0
94	0	.005	.005	0	0	0	0
95	0	.005	.005	0	0	0	0
96	0	.01	.01	0	0	0	0
97	0	.005	.005	0	0	0	0
107	0	.005	.005	0	0	0	0
108	0	.01	.01	0	0	0	0
109	0	.005	.005	0	0	0	0
110	0	.005	.005	0	0	0	0
111	0	.01	.01	0	0	0	0
112	0	.005	.005	0	0	0	0
113	0	.005	.005	0	0	0	0
114	0	.01	.01	0	0	0	0
115	0	.005	.005	0	0	0	0
125	0	.005	.005	0	0	0	0
126	0	.01	.01	0	0	0	0
127	0	.005	.005	0	0	0	0
128	0	.005	.005	0	0	0	0
129	0	.01	.01	0	0	0	0
130	0	.005	.005	0	0	0	0
131	0	.005	.005	0	0	0	0
132	0	.01	.01	0	0	0	0
133	0	.005	.005	0	0	0	0
143	0	.005	.005	0	0	0	0
144	0	.01	.01	0	0	0	0
145	0	.005	.005	0	0	0	0
146	0	.005	.005	0	0	0	0
147	0	.01	.01	0	0	0	0
148	0	.005	.005	0	0	0	0
149	0	.005	.005	0	0	0	0
150	0	.01	.01	0	0	0	0
151	0	.005	.005	0	0	0	0
161	0	.005	.005	0	0	0	0
162	0	.01	.01	0	0	0	0
163	0	.005	.005	0	0	0	0
164	0	.005	.005	0	0	0	0
165	0	.01	.01	0	0	0	0
166	0	.005	.005	0	0	0	0
167	0	.005	.005	0	0	0	0
168	0	.01	.01	0	0	0	0
169	0	.005	.005	0	0	0	0
179	0	.005	.005	0	0	0	0
180	0	.01	.01	0	0	0	0
181	0	.005	.005	0	0	0	0
182	0	.005	.005	0	0	0	0
183	0	.01	.01	0	0	0	0
184	0	.005	.005	0	0	0	0
185	0	.005	.005	0	0	0	0
186	0	.01	.01	0	0	0	0
187	0	.005	.005	0	0	0	0
197	0	.005	.005	0	0	0	0
198	0	.01	.01	0	0	0	0
199	0	.005	.005	0	0	0	0
200	0	.005	.005	0	0	0	0