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ANALYSIS AND DESIGN OF TRANSVERSE, STAGGERED WALL-BEAMS

by

DILIP G. NAMJOSHI

B. E. (HONORS) Civil Engineering, 1969

S. P. College of Engineering, The University

of Bombay, India

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Approved by

Stuart E Swartz

MAJOR PROFESSOR

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GENERAL NOMENCLATURE

A = area

A_g = gross sectional area of concrete

A_s = area of main steel reinforcement

A_u = area of upper framing flange

b = width of upper framing flange

C = total compressive force

d^0 = effective depth of the beam

d = overall depth of the beam

E_c = modulus of elasticity for concrete

E_s = modulus of elasticity for steel

f = arm of the primary couple

I_x = moment of inertia of the beam at any section x

I_u = moment of inertia of UFF

LFF = lower framing flange

L = length

M^0 = external bending moment

M' = primary couple

M'' = moment in UFF

m = bending moment for $T = 1$

n = modular ratio E_s/E_c

N = $M'_{(max)}/f$

P_T = axial force due to T

p = axial force, P_T when T = 1

T = total tensile force

UFF = upper framing flange

V = shear force

V_u = ultimate shear force

v_u = ultimate shear stress

v_c = Allowable shear stress

W = load

x = dimensionless number

α = $\left(\frac{\text{width of an opening}}{L}\right)$

γ = $\left(\frac{\text{distance of an opening center line from left support}}{L}\right)$

ϕ = capacity reduction factor

ϕ_1 = slope at the left end of the beam

ϕ_2 = slope at the right end of the beam

INTRODUCTION

Wall-beams - staggered and transverse are used in a new structural framing system which can be used in the design of high-rise reinforced concrete buildings. The conventional structural framing systems used to-day are flat plate and shear wall. The wall-beam system is an attractive and possibly economical alternative to the present framing systems. Although the system was originally conceived for structural steel construction (1)*, the basic concepts can be applied to reinforced concrete buildings also.

The wall-beam system was introduced by Portland Cement Association, Skokie, Illinois (2). The system was the result of efforts made to find an efficient way to resist wind load and yet have a design which is economical and flexible.

This report discusses the architectural and structural concepts involved in the new system. The theory and analysis are presented to introduce the new system. A design example of a typical wall-beam in a high-rise building using a wall-beam system is also presented in this report.

The wall-beam, as a principal component of the wall-beam framing system is discussed in the report. The detailed study of the wall-beam framing system as a whole is very complicated. However, the study of an isolated wall-beam, along with its analysis and design is useful in giving an introductory idea to the new framing system.

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*Numbers in parentheses refer to items listed in Bibliography.

REVIEW OF LITERATURE

The Departments of Architecture and Civil Engineering, Massachusetts Institute of Technology prepared a report (1) which introduced a new concept in structural steel framing systems. The report showed that for high-rise structural steel buildings a reduction in overall cost could be achieved. The reduction in the cost of the structure was possible because the system offered an efficient way to resist the wind load. The new system consisted of story-high Pratt-type trusses with staggered locations on each floor level. The report stated that if the new framing system is used, 50% of the structural steel can be saved as compared to the conventional system of braced frames. This system does not affect the planning flexibility of a structure.

The Portland Cement Association, Skokie, Illinois conducted research (2) to determine the applicability of the new framing system to reinforced concrete buildings. The research included an analysis and a design of a concrete wall-beam with rectangular web openings. An experimental investigation was also conducted on a half scale model of a wall-beam.

The Portland Cement Association then developed a digital computer program for the analysis and design of the wall-beam framing system for use on the IBM 1130 computer (3).

ARCHITECTURAL DESIGN

High-rise residential buildings and commercial buildings are an essential part of the major cities in the U.S. The wall-beam system can be effectively used in many types of high-rise structures such as apartments having permanent interior partitions, student dormitories, hotels, etc.

It is a difficult task to explore the architectural implications of the wall-beam framing system in completely general terms. Apartment planning and construction are governed by state regulations or local city regulations. Therefore, a typical apartment building which is somewhat idealized is considered here to demonstrate the merits of this new system. The wall-beam system is suitable primarily for a rectangular plan of an apartment building but it can also be effectively used in other types of plans such as cruciform, Y-shape, broken rectangle etc. as long as the clear spans of the wall-beams extend from outside to outside of the building.

The building illustrated in Fig. 1. has the floor slabs with continuous 12'0" spans (since the slabs are either supported or suspended at 12 - ft. intervals) and 24'0" spaces between two adjacent wall-beams on any floor level.

Within certain limitations the wall-beam system provides for very flexible planning of an apartment unit if the basic requirements are not violated. Many variations are possible in a wall-beam building layout to accomodate desired features of the architectural planning. In each layout, the wall-beams occur at every second floor on each column line. Due to the staggered locations of the wall-beams, a wall-beam building will always have two typical floors. The layout of any typical floor is repeated two floors above and two floors below.

Various types of possible layouts in a wall-beam system are possible. With some alternation in a regular wall-beam layout it is possible to make changes at various levels within the height of the building. Using different wall-beam layouts at various levels is advantageous if the type of apartments in the upper floors differs from that in the lower floors. This is an interesting fact about the planning flexibility of the wall-beam system. The shear wall or flat plate construction does not allow such variation which can be conveniently accomodated in a wall-beam building.

The equipment for heating, plumbing and electrical services can be conveniently located adjacent to the corridor walls. If the architectural planning allows, they could be located anywhere within the apartment units. In the wall-beam system floor plans for two adjacent floors differ and hence more quantity of plumbing fixtures is required because of the vertical nonalignment of bathrooms and kitchens. However this has a negligible effect on the total cost of the structure.

Elevator shafts and stairwells can be located in the wall-beam system without any considerable alteration in the structural planning. Stairwells can be located at the two ends of the corridor. The elevator shafts can be located adjacent to the corridor. In the buildings of moderate heights the walls of elevator shafts and stairwells can be made of non-load-bearing masonry. However in high buildings the walls can be of load-bearing reinforced concrete. They would stiffen the building both longitudinally and transversely.

STRUCTURAL DESIGN

The structural behavior of the wall-beam framing system will be studied under two different loadings; i.e., i) Gravity loading and ii) Lateral loading.

i) GRAVITY LOADING:

Gravity loading consists of the dead load, live load, partition load etc.

The wall-beam spanning across the entire width of the building acts as an I-section, the upper and lower slabs forming the upper flange and the lower flange respectively. For an isolated wall-beam, the upper framing flange (UFF) forms the compression flange and the lower framing flange (LFF) forms the tension flange [Fig. 2.].

The floor slab which is supported over two wall-beams at the ends and suspended from a wall-beam at the center demonstrates an interesting phenomena. Outside strips of such a floor slab each having a width one-fourth of the total span form the compression flanges of the wall-beams that are supporting the slab. The middle strip of a width equal the half-span forms the tension flange of the wall-beam from which the slab is suspended. Thus the floor slab has different structural behavior in three zones, two at the ends and one in the middle. The slab is cast monolithically and therefore it transfers shear force at the junction of adjacent compression and tension zones parallel to the wall-beam. This longitudinal shear transfer through the floor slab is a unique phenomenon in the wall-beam system.

The wall-beam is designed on the basis of a cracked section, thus neglecting the tension carrying capacity of the concrete. The wall-beam has a large moment of inertia as compared to the supporting columns. The effect of wall-beam end moments is thus neglected and the wall-beam is designed as a 'simple beam with web openings'. Most wall-beams will have web openings which will tend to reduce the shear capacity of these wall-beams. It may be assumed that the portion of a beam over the web opening will carry all the vertical shear. The gravity loading creates a combined bending and an axial compression in this portion of the beam.

For ease of construction and to satisfy the minimum thickenss requirement, a 6-in. web is recommended. This thickness permits one layer of reinforcing and adequate sound insulation. Thicker webs may be provided if the loading conditions require them.

The rectangular web openings cause a stress concentration around the corners (8)(9). The stress concentration must be considered in the design procedure.

The construction sequence will involve casting floor slabs and webs separately. Slabs contain the main tensile steel for the wall-beams. The slab forming the LFF is suspended from the beam web and the interface between web and flange is subjected to some tension and horizontal shearing force. The capacity of the interface should be checked for the safety.

ii) LATERAL LOADING:

Behavior of the wall-beam system under the influence of lateral load is complex. Adjacent bents in this system have different characteristics.

Considering an isolated bent, its behavior under lateral loading can be easily visualized. Assuming the slabs to have an *infinite axial stiffness*, all points at any particular level must have equal horizontal displacement. This means that a bent would undergo the deformation in 'stiff beam-flexible column' manner. This deformation pattern will not occur in the wall-beam system because of the different characteristics of the adjacent bents. This deformation pattern does not take into account the effect of a bent on the adjacent bent; when two adjacent bents are considered together, the points at the same level do not have the same horizontal displacements [Fig. 3(a)]. Therefore, a bending pattern must be assumed which is consistent with the structural theory and behavior. Such a pattern is assumed to have equal horizontal displacements at any floor level and the column bending is a 'single-curvature' type, as in the case of a cantilever beam [Fig. 3(b)].

The horizontal shear due to the lateral loading is resisted by a wall-beam in the manner demonstrated in Fig. 4(a). Axial loads are induced in the columns supporting the wall-beam. The floor slabs acting as rigid diaphrams, transfer the horizontal shears from the wall-beams on one floor to the wall-beams on adjacent column lines on the floors above and below [Fig. 4(b)]. A sketch of the forces acting on a wall-beam which has been isolated from the rest of the structure is shown in Fig. 5(a).

The presence of web openings will cause the wall-beam to deform as shown in Fig. 5(b) with the overall deflected shape similar to Fig. 5(c). The building may be analyzed by combining adjacent frames.

The wall-beams with unsymmetrical web openings have different rotational stiffnesses at each end. The restraint provided by the adjacent wall-beams prevents lateral deformation of the columns in each of the above composite frames. The frames necessarily act as cantilevers.

Several alternatives are possible for lateral loads applied in the longitudinal direction. The outside columns with the floor slabs constitute a frame sufficient to resist the horizontal loading for a building of moderate height. Shallow spandrel beams at the column lines may stiffen the frame in case of higher buildings. Another alternative is to provide shear walls in the elevator shaft or stair well. In this case the analysis should take into account the interaction of the shear wall and frame system (5).

STRUCTURAL PLANNING

The basic structure in a wall-beam building is simple and repetitive. The structural planning may change to suit the requirement of a building. Many times it is preferable to make the walls of the elevator shaft or stairwell non-load-bearing masonry. Then the interior of the lower floors can be supported on the floor slab or on shallow beams. However, in case of higher buildings, load bearing reinforced concrete walls prove to be of advantage because they stiffen the structure longitudinally and transversely and decrease the shear induced due to lateral loading. They might eliminate the necessity of spandrel beams along the sides. In this case, however, completely clear areas in the lower floors would not be possible.

Exposed columns should be avoided because they present a difficult design problem. Considerable research work has been done to develop methods of analysis for exterior exposed columns having a large degree of exposure (4). If the columns act as vertical load carrying members in a wall-beam structure, the effect of temperature stresses can be neglected because the entire building will shorten or elongate freely with the changes in temperature. The presence of interior bearing walls may result in different thermal movements of exposed columns.

Rectangular exterior columns appear to be the best choice considering both architectural and structural points of view. Column lines should preferably have their longitudinal axis parallel to the long dimension of the building. Such an orientation of columns improves longitudinal wind resistance.

Since exterior columns are closely spaced, some footing cost can often be reduced if continuous strip footings are used throughout the length of the building.

ANALYSIS AND DESIGN OF A WALL-BEAM

i) INTRODUCTION:

The wall-beam under consideration is an I-section consisting of a story-high wall acting as a web and floor slabs forming the top and bottom flanges. The portion of the wall-beam above the web opening (generally a T-section) will be defined as the UFF, i.e., Upper Framing Flange. The portion of the wall beam (generally rectangular in section) forming the lower edge of the web opening will be defined as the LFF, i.e., Lower Framing Flange [Fig. 2.].

The wall-beam will have openings in the web for corridors, doors or windows. The principal effect of an opening is to reduce the shear capacity of the beam at that section. The reduction in the shear capacity results in an increase in the deflection. For this precise reason, openings in the zone of high shear in the wall-beam are avoided. The web opening for a corridor is generally centrally located where the shear is negligible.

The design procedure follows essentially the same procedure used for conventional reinforced concrete beams. Special design considerations are required for the shear capacity of the weaker sections of the wall-beam.

In the analysis of forces acting at the end of the wall-beam, the entire frame is considered. In the upper floors, the wall-beam is stiffer than the supporting columns. In this report the wall-beam is designed as a simple beam for the purpose of gravity load design. In the lower floors the columns are heavier and the wall-beam is designed as a beam framing into columns. The latter case involves analysis of an H-frame by the moment distribution method. In this analysis, stiffness factors and distribution factors are required. Unsymmetically located openings have different rotational stiffnesses at each end.

Under lateral loading, a wall beam-frame acts as a vertical cantilever. The relative lateral displacement of the floors is due only to axial deformation of the columns (the lateral or shear deformation of the columns is restrained by the wall-beam in the adjacent frames.) The axial deformation produced in the columns due to lateral loads induces moments and shears in the wall-beams. Analysis of such a frame can be carried out by a digital computer program (3).

Although most wall-beams may be analyzed for gravity loads by assuming simple supports at the end, the more general case involving the end moments is treated in the analysis. The problem is considered as a superposition of two cases, namely,

(a) A simply-supported wall-beam under the given vertical loads.

(b) A simply-supported wall-beam subjected to the calculated end moments as obtained from the frame analysis, and the proper equilibrating end shears.

ii) ANALYSIS OF A SIMPLY-SUPPORTED BEAM WITH A RECTANGULAR OPENING IN THE WEB UNDER VERTICAL LOADING.

Forces and moments acting on the UFF and LFF spanning the opening will be determined by this analysis. The forces and moments acting on the solid web section beyond the opening are taken equal to the values that would occur if it were a solid web beam.

A basic assumption in the analysis states that the LFF acts mainly as a tension member.

The external bending moment, M_x^0, at any section through the opening is given by the expression

$$M_x^0 = M' + M_x'' \tag{1}$$

where,

M' = the primary couple formed by the tension T in the LFF and the compressive force C in the UFF.

M_x'' = the moment in the UFF.

This assumption can be easily justified because the LFF is a thin slab cracked in tension acting like a tie bar while the UFF is a T-section subjected to combined axial force and flexure. The beam can be viewed as a tied rectangular frame with very stiff legs.

Rearranging Eq. (1),

$$\begin{aligned} M_x'' &= M_x^0 - M' \\ &= M_x^0 - T \cdot f \end{aligned} \tag{2}$$

where

T = the tensile force in LFF (Fig. 6).

f = the arm of the primary couple.

M_x^0 = the simple beam bending moment at section 'X' through the opening.

For a known value of T, M''_x can be calculated using Eq. (2). The analysis is done using the influence line for T.

Influence Line for Tension, T. in the LFF

Using the basic moment area principles or conjugate beam theory, the end slopes of a beam due to force T acting at the assumed cut end can be calculated. By the principle of virtual work, the relative displacement of the assumed cut end of the LFF due to tension T is given by

$$\delta_T = \int_0^1 \frac{M_{xT} \cdot m_t \cdot L \cdot dx}{E_c \cdot I_u} + \frac{P_T \cdot p \cdot \alpha L}{E_c A_u} + \frac{P_T \cdot p \cdot \alpha L}{E_s A_s} \tag{3}$$

where

M_{xT} = the bending moment at any section due to tension T.

m_t = the bending moment M_{xT} when T = 1.

P_T = the axial force in the framing flange due to T.

p = axial force in the framing flange due to T = 1

In Eq. (3), x is used as a dimensionless ratio, the distance from the left support to a general section being given by xL. [Fig. 6(a)]

The bending moment M_{xT} is equal to Tf over the length of the UFF and zero elsewhere. The axial force P_T is equal to T for the LFF and C = -T for the UFF. Taking T = 1, and assuming that the concrete in the LFF carries no tension, Eq. (3) becomes

$$\delta_T(T=1) = \int_{(\gamma-\frac{\alpha}{2})}^{(\gamma+\frac{\alpha}{2})} \frac{f^2(Ldx)}{E_c \cdot I_u} + \frac{L}{E_c A_u} + \frac{L}{E_s A_s} \tag{4}$$

where,

E_s = modulus of elasticity for the steel;

A_s = area of steel in LFF;

A_u = cross sectional area of the UFF + web above opening.

Eq. (4) becomes,

$$\delta_T(T=1) = \left[\frac{f^2 Lx}{E_c I_u}\right]_{(\gamma-\frac{\alpha}{2})}^{(\gamma+\frac{\alpha}{2})} + \frac{\alpha L}{E_s A_s} + \frac{\alpha L}{E_c A_u}$$

$$= \frac{\alpha L f^2}{E_c I_u} + \frac{\alpha L}{E_s A_s} + \frac{\alpha L}{E_c A_u}$$

$$= \frac{\alpha \cdot L}{E_c I_u}\left[f^2 + \left(\frac{I_u}{n \cdot A_s} + \frac{I_u}{A_u}\right)\right] \qquad (5)$$

where, n = the modular ratio $\frac{E_s}{E_c}$

The sum of the two terms in the parenthesis on the right hand side of Eq. (5) represents the combined effect of the axial deformation of the LFF and UFF, and is generally small as compared to the quantity 'f^2' which is associated with the flexural deformation of the UFF. For normal size wall-beams the term $\left(\frac{I_u}{n \cdot A_s} + \frac{I_u}{A_u}\right)$ ranges from 1.5 to 2.5 percent of the term 'f^2' (2).

Eq. (5) is therefore simplified to

$$\delta_T(T{=}1) = \frac{\alpha L f^2}{E_c \cdot I_u} \tag{6}$$

Using Eq. (6) the tension T required for $\delta_T{=}1$ is given by,

$$T = \frac{E_c I_u}{\alpha L f^2} \tag{7}$$

The ordinates of the deflection curve created in the beam by applying a displacement δ to the tie rod are proportional to the influence line for T by the Muller-Breslau principle. This deflection curve may be created by T as shown in Fig. 6(b).

Thus,

$$\phi_1 = (1-\gamma)\,\frac{\alpha L f}{E_c I_u} \cdot T$$

$$= (1-\gamma)\,\frac{\alpha L f}{E_c I_u} \cdot \frac{E_c I_u}{\alpha L f^2}$$

$$= \frac{1}{f}\,(1-\gamma)$$

In the above the value of T is such to make the displacement δ in the tie rod equal to unity.

Therefore, the ordinate of the influence line for T for $0 < x < (\gamma - \frac{\alpha}{2})$ is given by

$$\phi_1 \cdot xL = \frac{1}{f}\,(1-\gamma)\ xL. \tag{8a}$$

Referring again to Fig. 6(b),

$$T(\text{unit load at } xL) = \phi_1 \cdot xL - \frac{[T(\delta_T=1)]f}{E_c I_u} \times \frac{\{[x - (\gamma - \frac{\alpha}{2})]L\}^2}{2}$$

for $(\gamma - \frac{\alpha}{2}) < x < (\gamma + \frac{\alpha}{2})$.

Substituting $T = \frac{E_c I_u}{Lf^2}$ and $\phi_1 = \frac{1}{f}(1 - \gamma)$, the above equation becomes

$$T(\text{unit load at } xL) = \frac{L}{2\alpha f}\{-x^2 + 2x[(1 - \gamma) \times \alpha + (\gamma - \frac{\alpha}{2})] - (\gamma - \frac{\alpha}{2})^2\}$$

for $(\gamma - \frac{\alpha}{2}) < x < (\gamma + \frac{\alpha}{2})$. (8b)

Now for $(\gamma + \frac{\alpha}{2}) < x < 1.0$,

$$T(\text{unit load at } xL) = -\phi_2(1 - x)L$$

$$= \frac{1}{f}(1 - x)\gamma L. \tag{8c}$$

The complete influence line is shown in Fig. 6(b) and Fig. 6(c). The ordinate for the influence line for the primary couple, $M' = Tf$ can be obtained by multiplying the expressions in Eq. (8) by f. Thus,

$$M'(\text{for unit load at } xL) = (1 - \gamma)xL \cdots 0 \leq x \leq (\gamma - \frac{\alpha}{2});$$

$$M' = \frac{L}{2\alpha}\left\{-x^2 + 2[\alpha(1 - \gamma) + (\gamma - \frac{\alpha}{2})]\, x - (\gamma - \frac{\alpha}{2})^2\right\}$$

$$\cdots (\gamma - \frac{\alpha}{2}) \leq x \leq (\gamma + \frac{\alpha}{2});$$

$$M' = \gamma L(1 - x) \cdots (\gamma + \frac{\alpha}{2}) \leq x \leq 1.0. \tag{9}$$

Eqs. (8) and (9) depend upon the geometry of the wall beam, i.e. span, and location and width of the opening. These equations are independent of the section properties of the framing flanges.

The influence line for the total moment at section 'a' through the left edge of the opening is given by

$$M_a^0(\text{unit load at } xL) = (1 - \gamma + \frac{\alpha}{2})\ xL \cdots 0 \leq x \leq (\gamma - \frac{\alpha}{2})$$

$$M_a^0 = (\gamma - \frac{\alpha}{2})(1 - x)\ L \cdots (\gamma - \frac{\alpha}{2}) \leq x \leq 1.0 \qquad (10)$$

The moment at the left end of the UFF is then given by

$$M_a'' = M_a^0 - M'$$

where M' and M_a^0 can be obtained using Eqs. (9) and (10) respectively. This influence line is given in Fig. 6(d).

iii) ANALYSIS OF A SIMPLY SUPPORTED WALL-BEAM WITH A SINGLE RECTANGULAR OPENING IN THE WEB SUBJECTED TO A COUPLE ACTING AT ONE END (Solution for tension, T, in the LFF due to couple, M, acting at the left end of the beam)

By requiring consistent deformation, we can write

$$T = - \frac{\delta_T(\text{due to M at the end})}{\delta_T(T=1)}$$

where,

δ_T(due to M at the end) = relative displacement of the assumed cut ends of LFF due to M.

By the principle of virtual work

$$\delta_T(\text{due to M at end}) = \int_0^1 \frac{M_x \cdot m_t(Ldx)}{EI_x}$$

where

$$M_x = M(1-x)$$

and

$$m_t = \begin{cases} -1.f = -f \cdots (\gamma - \frac{\alpha}{2}) < x < (\gamma + \frac{\alpha}{2}); \\ 0 \qquad\qquad\qquad \cdots \text{ elsewhere.} \end{cases}$$

Therefore,

T(due to a couple M at the left end)

$$= \frac{M(1-\gamma)}{f} .$$

Corresponding to the above value of T,

$$M' = Tf = M(1 - \gamma);$$

$$M'' = M^0 - M' = M(\gamma - x) \cdots (\gamma - \frac{\alpha}{2}) \leq x \leq (\gamma + \frac{\alpha}{2}). \qquad (11)$$

Curves have been drawn for Eqs. (9), (10) and (11) to indicate an approach for analyzing forces in the framing flanges of the wall-beam with the web openings (2).

The secondary moment, M'', in the UFF is generally small compared to the primary moment M' = Tf when the opening is near the center of the beam. When the wall-beam is assumed to be simply supported, the longitudinal tensile reinforcement can be calculated from the gross external M^0 at that section with little loss in accuracy as shown in the design example.

DESIGN EXAMPLE

A wall-beam with a central rectangular web opening as shown in Fig. 7 will be designed.

Given: A wall-beam of span 66'-0".

web thickness = 6"

thickness of floor slabs = 5"

overall beam depth = 9'-4".

The opening is for a corridor 6'-0" wide and 7'-0" high. Assume the column sections above and below be 12"x24", the beam framing into the longer dimension of the column. Spacing of wall beams is 24'-0" o.c.

Steel ratio = 0.04

f'_c = 4000 psi

f_y = 60000 psi

Live load = 40 psf

Partitions = 15 psf of the floor area

Dead weight of corridor partitions = 25 psf of the wall area.

The design of a wall-beam under gravity loading only is presented.

A- Load on Girder:

Dead weight of the wall-beam and slab neglecting the opening

$$\frac{5(12+12) + 6(8.5)}{12\text{x}1000} \text{ x } 150 \qquad = 1.94 \text{ kips/ft.}$$

Partition load = (12+12) x 0.015 = 0.36 kips/ft.

Total uniform distributed load, $W_D = \overline{2.30 \text{ kips/ft.}}$

Live load (applying a 15 percent reduction due to the tributory area exceeding 300 sq. ft.) (6)

$$W_L = \frac{0.85\ (12+12)x40}{1000} = 0.82 \text{ kips/ft.}$$

Concentrated dead loads, P, acting at each edge of the opening due to the dead weight of corridor walls.

$$P = \frac{2x(12x8.5)x25}{1000} = 5.1 \text{ kips}$$

Design is done using ultimate strength design in accordance with the provisions of the ACI Building Code Requirements for Reinforced Concrete (ACI-318-63), hereinafter referred to as Code.(6)

As per Eq. (15-1) of Section 1506 of the code, the load factor is calculated.

Ultimate uniform distributed dead load

$$W_{UD} = 1.5x2.30 = 3.45 \text{ kips/ft.}$$

Ultimate uniform distributed live load

$$W_{UL} = 1.8x0.82 = 1.48 \text{ kips/ft.}$$

Total uniform distributed ultimate load

$$W_{UT} = 3.45 + 1.48 = 4.93 \text{ kips/ft.}$$

Ultimate concentrated dead loads, P_{UD}

$$= 1.5x5.1 = 7.65 \text{ kips.}$$

B- <u>Maximum external moment, $M^o(max)$ at a section through the center of the web opening</u>:

For the given opening

$$\alpha = \frac{6}{66} = 0.091$$

$$\gamma = \frac{33}{66} = 0.50$$

Due to uniform load W_{UT} over the entire span

$$M_W^o = \frac{4.93 \times (66) \times (66)}{8} = 2690 \text{ K-ft.}$$

Due to concentrated loads P_{UD} at the edges of the opening

$$M_P^o = (7.65x33) - (7.65x3) = 228.5 \text{ K-ft.}$$

$$M^o_{(max)} = M_W^o + M_P^o$$

$$= (2690 + 228.5) = 2918.5 \text{ K-ft.}$$

External bending moment, M^o, at the left edge of the opening is given by,

$$(170.3)(30) - \frac{(4.93)(30)^2}{2}$$

$$= 2889 \text{ K-ft}$$

Assume the value of the primary arm 'f' to be equal to the distance from C.G. of the upper slab to the C.G. of the main reinforcement in the lower slab. This assumption neglects the capacity of web to carry compression.

Therefore,

$$f = 8'\text{-}6''+5''$$

$$= 107''$$

C- Calculation for maximum shears at supports and edges of the opening.

Maximum shear at the support

$$V_{s(max)} = (4.93 \times 33) + 7.65 = 170.3 \text{ Kips.}$$

Maximum shear at the left edge of the opening,

$V_{1(max)}$ due to uniform dead and concentrated loads

$$= (3.45 \times 3) + 7.65$$

$$= 18.0 \text{ kips}$$

$V_{2(max)}$ due to uniform live load over the beam from the right of the left edge of the opening.

$$= \frac{1.48 \times 39 \times 39}{2 \times 66} = 17.1 \text{ kips}$$

Total shear $V_{(max)} = V_{1(max)} + V_{2(max)}$

$$= 18.00 + 17.1$$

$$= 35.1 \text{ kips.}$$

The maximum shear at the right edge of the opening is also 35.1 kips by reason of symmetry.

Design of Web Reinforcement for the portion over the opening

The maximum shear stress carried by an unreinforced web is given by

[Equation following Eq. (17-3) of code]

$$v_c = 3.5 \cdot \phi\sqrt{f'_c(1 + 0.002\ N/Ag)}$$

where, $N = \frac{M'}{f}$ at left edge $= \frac{2889 \times 12}{107} = 324$ kips

Ag = Gross sectional area of concrete.

The width of the UFF is calculated using section 906(b) of code.

The width of the UFF, b = (2x8x5) + 6

= 86"

$Ag = (86x5) + (18x6) = 538\ in^2$

$$v_c = (3.5) \cdot (0.85) \sqrt{4000(1 + 0.002 \cdot \frac{324000}{538}}$$

$$= 279 \text{ psi}$$

Shear stress at the left edge of the opening,

$$v_u = \frac{V_{(max)}}{bxd} = \frac{35100}{bxd} = \frac{35100}{6x18} = 325 \text{ psi}$$

Section 1705 of Code limits the calculated shear stress, v_u, to

$10\phi\sqrt{f'c}$ or $10x0.85 \times \sqrt{4000} = 538$ psi > 325 psi if stirrups are present

Therefore,the web thickness need not be revised.

Assuming No. 3 U-Stirrups

Required spacing, $s = \frac{0.85 \times 2 \times 0.11 \times 60,000}{6(325 - 279)}$

$= 41.6"$

Since the maximum stirrup spacing is limited by Section 1706(a) of Code to a value of $\frac{d}{2}$ or 9", use, for the UFF, No. 3 U-stirrups spaced at 9" O.C. [Fig. 7(b)].

<u>Stirrup Spacing in Solid-Web Portion of the Wall-Beam</u>

At a section distant from the support the effective depth of the wall beam, d^o

= approx. 110 in.

$$V_u = 170.3 - \frac{110 \times 4.93}{12}$$

$$= 126 \text{ kips.}$$

The corresponding shear stress at the above section, $v_u = \frac{V_u}{b'd^o}$

$$= \frac{12600}{6x110} = 189.2 \text{ psi.}$$

Using Section 1701(b) of the Code, the shear stress, v_c, carried by an unreinforced web is given by $2\phi\sqrt{f'_c}$ or $2x0.85 \times \sqrt{4000} = 107.5$ psi.

Assuming a single line of No. 3 bar vertical reinforcement

$$\text{Required spacing, } s = \frac{0.85 \times 1 \times 0.11 \times 60{,}000}{6(189.2 - 107.5)}$$

$$= 11.5"$$

The section at which $v_c = v_u$ can be calculated. No vertical shear reinforcement is required beyond this section.

Referring to shear force diagram [Fig. 8] the distance of the required section from the left support, x is calculated

$$71.0 = 170.3 - 4.93 \times (x')$$

$$x' = 20.2 \text{ ft} = \text{approx. } 20'\text{-}3''$$

Section 1706(b) of the code provides the minimum cross-sectional area of web reinforcement per foot, $A_s = 0.0015b' = 0.0015 \times 6 \times 12 = 0.108 \text{ in}^2$ Maximum allowable spacing of the vertical web reinforcement corresponding to the above minimum required area

$$S_{max} = \frac{12}{0.108/0.11} = 12.3 \text{ in}$$

A 11-in. spacing of No. 3 bars along the solid-web portion is used [Fig. 9(a)].

D - Principal Longitudinal Reinforcement in the LFF

For a wall-beam with a web opening at the center and a small value of α, usually $\alpha \leq 0.1$ the secondary moment M''_x can be neglected. In this case the primary bending moment M'_x is approximately equal to the external bending moment, M^o_x. This can be proved as follows.

Using Eq. (9) and the symmetry of the wall beam and loading

$$M'_{(max)} \text{ at the center} = 2 \int_0^{L(\gamma-\frac{\alpha}{2})} (1 - \gamma) \cdot w \cdot x'dx'$$

$$+ 2 \int_{L(\gamma-\frac{\alpha}{2})}^{\frac{L}{2}} \frac{L}{2\alpha} \cdot w \cdot \left\{- \frac{x'^2}{L^2} + 2[\alpha(1-\gamma) + (\gamma - \frac{\alpha}{2})] \frac{x'}{L} - (\gamma - \frac{\alpha}{2})^2\right\}dx'$$

where, $x' = x.L$

using $\gamma = 0.5$ we get,

$$M'_{(max)} = 2 \int_0^{\frac{1}{2}(1-\alpha)} (\tfrac{1}{2})\ wx'dx'$$

$$+ 2 \int_{\frac{L}{2}(1-\gamma)}^{\frac{L}{2}} \frac{L}{2\alpha} \cdot w \left\{- \frac{x'^2}{L^2} + 2[\alpha \cdot \frac{1}{2} + \frac{1}{2} - \frac{\alpha}{2}]\ \frac{x'}{L} - (\frac{1}{2} - \frac{\alpha}{2})^2\right\} dx'$$

$$= w \int_0^{\frac{L}{2}(1-\gamma)} x'dx' + w \int_{\frac{L}{2}(1-\gamma)}^{\frac{L}{2}} \frac{L}{2\alpha} \left\{- \frac{x'^2}{L^2} + \frac{x'}{L} - (\frac{1}{2} - \frac{\alpha}{2})^2\right\} dx'$$

integrating,

$$M'_{(max)} = \frac{wL^2}{8}\ (1 - \alpha)^2 + \frac{wL^2}{8}\ (2\alpha - \frac{4}{3}\ \alpha^2)$$

$$= \frac{wL^2}{8}\ [1 - \frac{\alpha^2}{3}]$$

for $\alpha = 0.1$

$$M'_{(max)} = \frac{wL^2}{8}\ [1 - \frac{0.01}{3}]$$

$$= \frac{wL^2}{8}\ [0.9967]$$

The external bending moment for a solid beam at center is $\frac{wL^2}{8}$. Therefore in the actual design procedure the external bending moment $M^o_{(max)}$ is used instead of the primary bending moment $M'_{(max)}$ without much loss of accuracy.

For the general case of a wall-beam, curves can be drawn for different values of α giving the primary bending moment as a product of external bending moment and some factor. The proof given above is also true for a beam loaded with concentrated loads.

Assuming the line of action of the resultant compressive force in the UFF to be coincident with the centroid of gross concrete section, the reinforcement in the Lower Framing Flange is computed as follows.

$f = 107$ in

$\phi = 0.90$ .... capacity reduction factor

A_s, area of steel required,

$$A_s = \frac{M^o_{(max)}}{\phi \cdot f \cdot fy}$$

$$= \frac{2918.5 \times 12}{0.90 \times 107 \times 60}$$

$$= 6.06 \text{ sq. in.}$$

Use 5 No. 10 bars. (A_s furnished = 6.15 sq. in.) The bars will be arranged in two tiers [Fig. 7(b)]. Two bars in the top tier will be curtailed at a distance of 9'-0" from the support. Details of reinforcement are shown in Fig. 9(a).

<u>E</u> - Design of reinforcement around the web opening.

Additional reinforcement will be provided around the web opening to resist the diagonal tension and stress concentration at the corners of the opening.

A stress concentration factor of 2 will be used (8).

Shear force at the left edge of the opening = 35.1 kips.

Using the stress concentration factor of 2, the design shear force

= 2 x 35.1

= 70.2 kips.

The bars will be bent at 45° to the vertical

Therefore the bars must resist a force of

$$\sqrt{2} \times 70.2 \text{ kips} = 99 \text{ kips.}$$

$$A_s = \frac{99}{60} = 1.17 \text{ sq. in.}$$

Use 1 No. 10 bar. (A_s furnished 1.23 sq. in)

For practical considerations the same reinforcement will be used at all the corners of the opening.

For tying the stirrups 2 No. 5 bars will be used in the upper framing flange along the entire length of the beam.

At the bottom of the portion of the wall-beam over the web opening 2 No. 5 bars will be used to tie stirrups.

2 No. 5 bars will be used along the edges of the opening to provide additional reinforcement.

No. 3 bars at 12" o.c. will be used along the length of the beam to resist the buckling of vertical stirrups.

E - Calculation of Midspan Deflection Under a Uniformly Distributed Load Over Entire Span.

$$W_T = W_D + W_L = 2.30 + 0.82$$

$$= 3.12 \text{ kips/rft.}$$

Using equivalent uniformly distributed load for the concentrated loads

$$W = 3.12 + 0.15$$

$$= 3.27 \text{ kips/ft.}$$

$$\delta = \frac{5WL^4}{384EI} = \frac{5(3.27)(66)^4}{384(3x10^3)I}$$

where

I = moment of inertia of the gross section.

Calculation for I

Effective upper flange width = 86 in.

A width equal to 12 in is assumed for lower flange.

Area of section = (86 x 5) + (102 x 6) + (12 x 5)

$$= 1102 \text{ in}^2$$

Distance of neutral axis from the top edge of the section

$$\bar{Y} = \frac{430\ (2.5) + 612\ (56) + 60\ (109.5)}{1102}$$

$$= 38 \text{ in}$$

Therefore,

$$I_{NA} = \frac{86(5)^3}{12} + 430\ (38 - 2.5)^2 = 542900 \text{ in}^4$$

$$+ \frac{6x(102)^3}{12} + 612\ (38 - 51)^2 = 626200 \text{ in}^4$$

$$+ \frac{12x(5)^3}{12} + 60\ (112 - 40.5)^2 = \underline{307100 \text{ in}^4}$$

$I_{NA} = 1.476.200 \text{ in}^4$

Hence,

$$\delta = \frac{5(3.27) \times (66)^4 \times 1728}{384 \times 3 \times 10^3 \times 1.476 \times 10^6}$$

$$= 0.313 \text{ in.}$$

It is recommended that the wall-beams be cambered to allow for estimated deflection under long sustained loads.

CONCLUSIONS

Planning advantages and economics are the governing factors in the choice of structural framing systems. The wall-beam system shows advantages over the conventional beam-column system in the area of planning flexibility. These are:

1. More variations in the structure at various levels within the height of the building are possible to accomodate changes in the layout.
2. Larger floor areas are available by eliminating wall-beams on the lower floors. Such areas provide a clear space from outside to outside of a building. In the beam-column system such clear areas have columns that cannot be eliminated.
3. Better horizontal sound insulation between apartments is possible.
4. As in the beam-column system the designer can adhere to a standard module. Prefabrication of plumbing and heating components, interior walls, glass work etc. is feasible because of the regularity in the structure.

Some planning disadvantages of this framing system can not be neglected. The disadvantages are given below:

1. It is not possible to avoid locating one type of room over another type so planning must be restricted somewhat in order to align service shafts in case of kitchen and bathroom areas.
2. Though the wall-beam system is flexible, apartment layouts are restricted by the structure, and hence the planning is more confined than flat plate construction where apartment layouts are more or less unrestricted by the structure.

Discussion of Design Example

The design example presented in this report deals with a simple case of a typical wall-beam. The theory and analysis presented in this report treat the general case of the wall-beam. Using the theory and analysis, design of a more complicated wall-beam should not prove difficult.

As the design example is intended to present a general approach to a design procedure, the problem selected is simple in nature. The equations derived are modified to suit the problem.

Design of floor slabs is not considered because it can be done using standard design procedure. A floor slab reinforced in two directions can resist the longitudinal shear force transmitted through the floor slab.

It is often possible to have a simple configuration of a wall-beam and hence the design example can be a useful design aid.

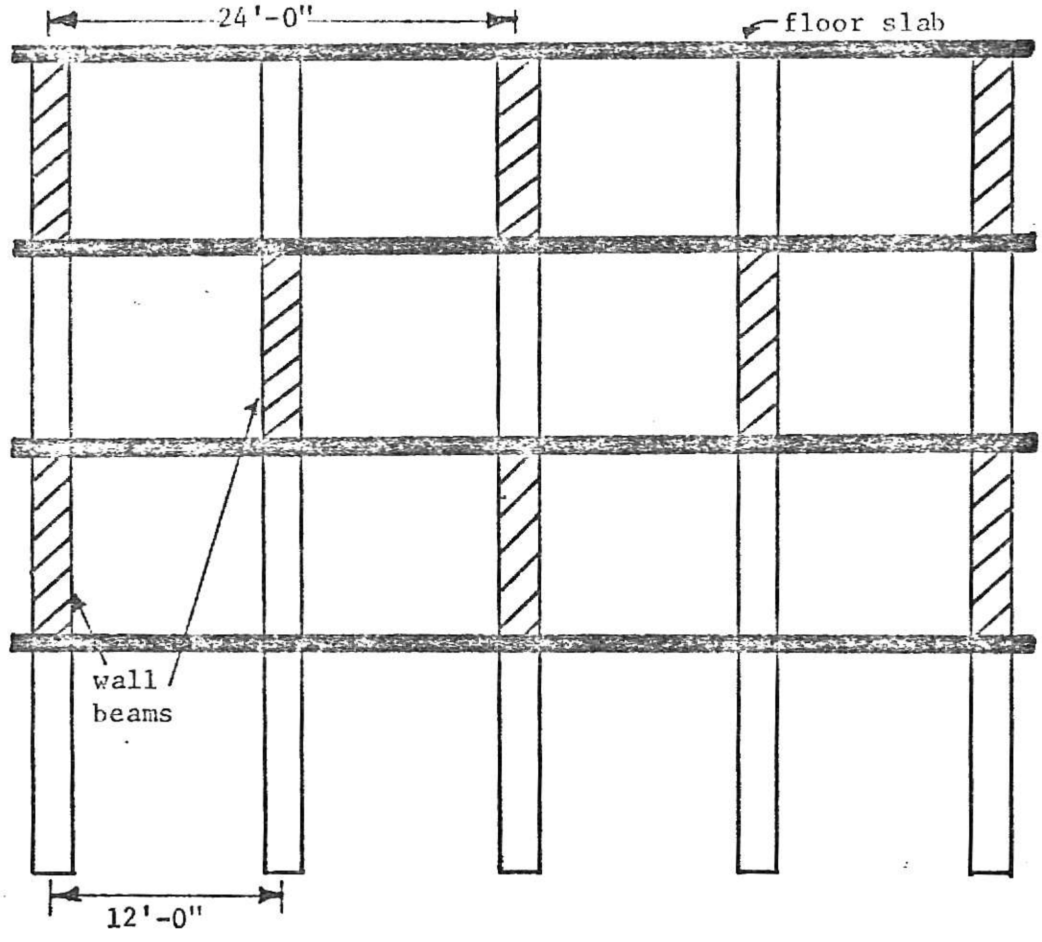


Fig. 1 A Typical Wall-Beam Layout

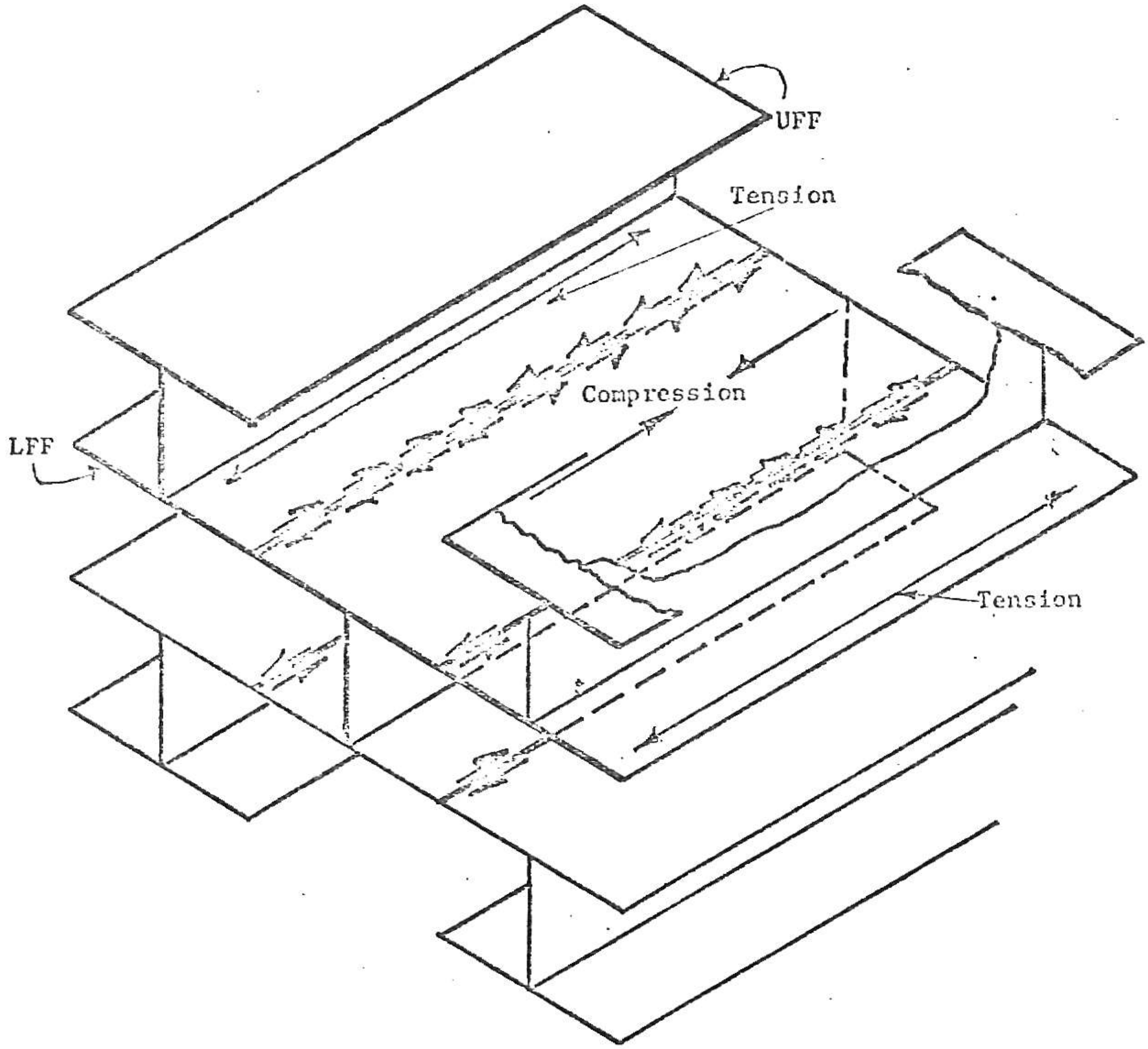
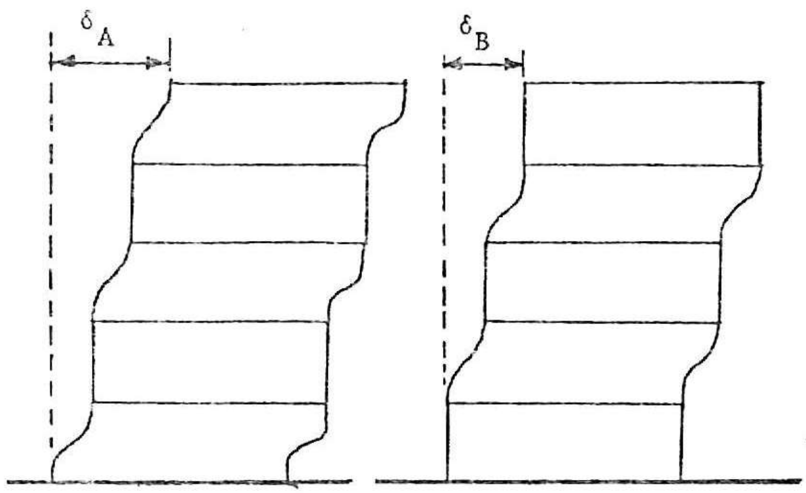
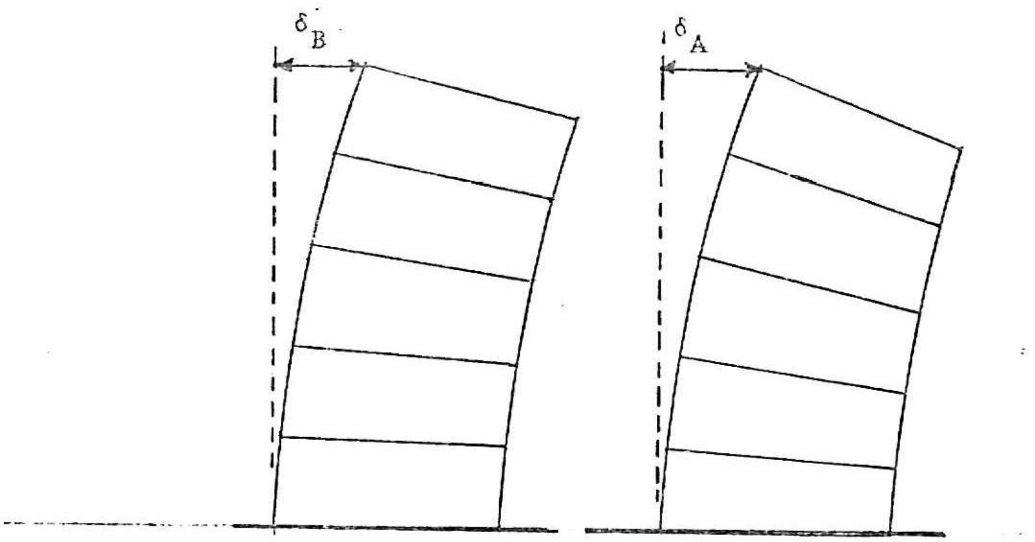


Fig. 2 Wall-Beam Action Under Gravity Loads



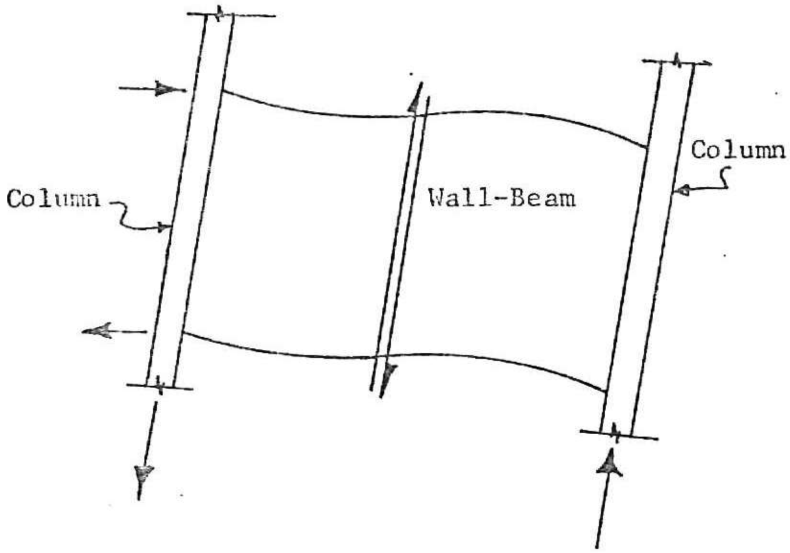
(a)

Wall-Beam Action Under Lateral Loads for Bents Acting Separately



(b)

Fig. 3 Lateral Action of Wall-Beam System, if Adjacent Bents Act Separately or if Adjacent Bents Act Together



(a)

Deformation of Typical Wall-Beam Unit
Under Lateral Loads

(b)

Shear Transfer Under
Lateral Loads

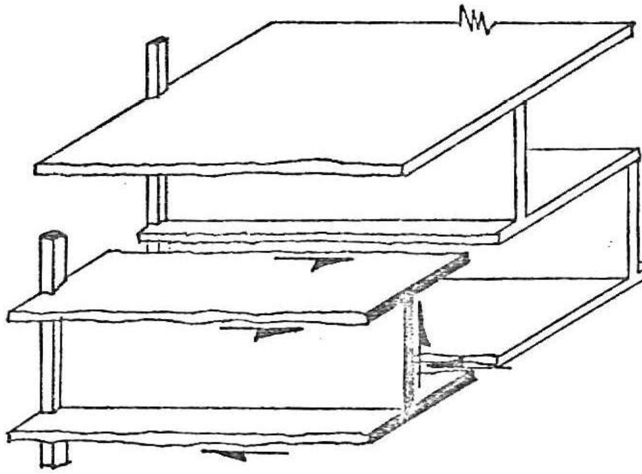


Fig. 4 Wall-Beam Action Under Lateral Loads

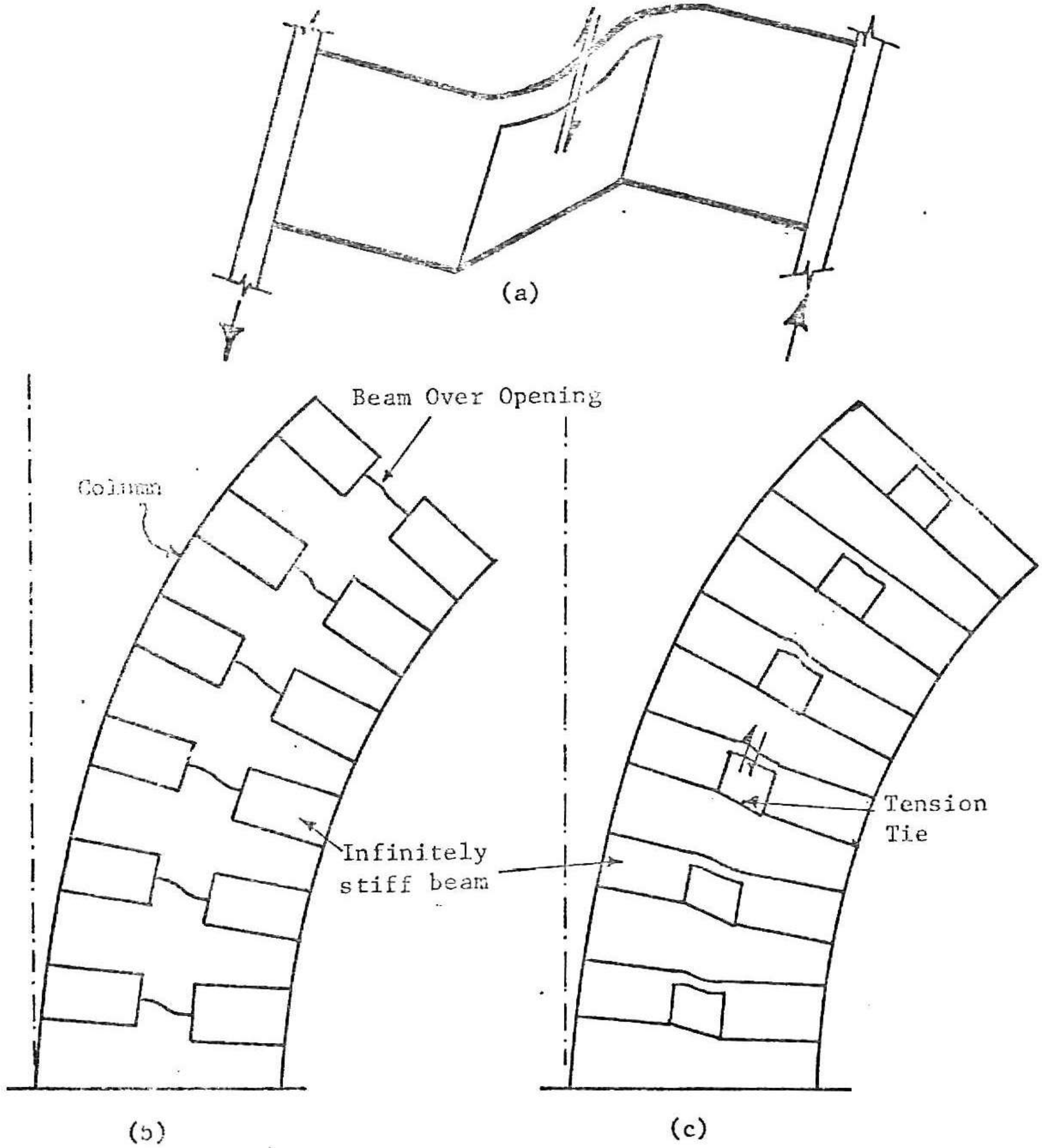


Fig. 5 Action of Wall-Beams with Openings Subjected to Lateral Loads

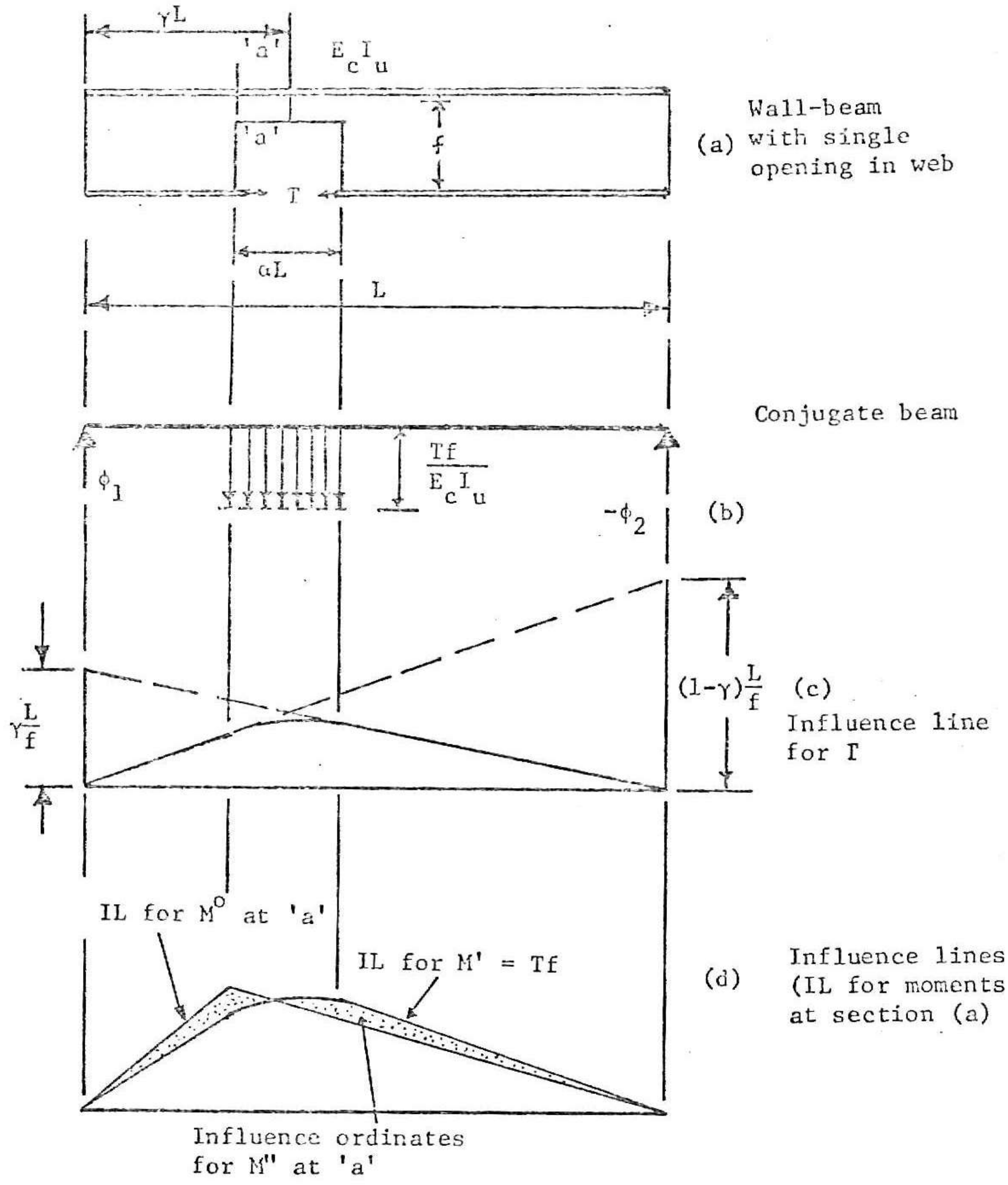


Fig. 6 Analysis of a Wall Beam with Opening

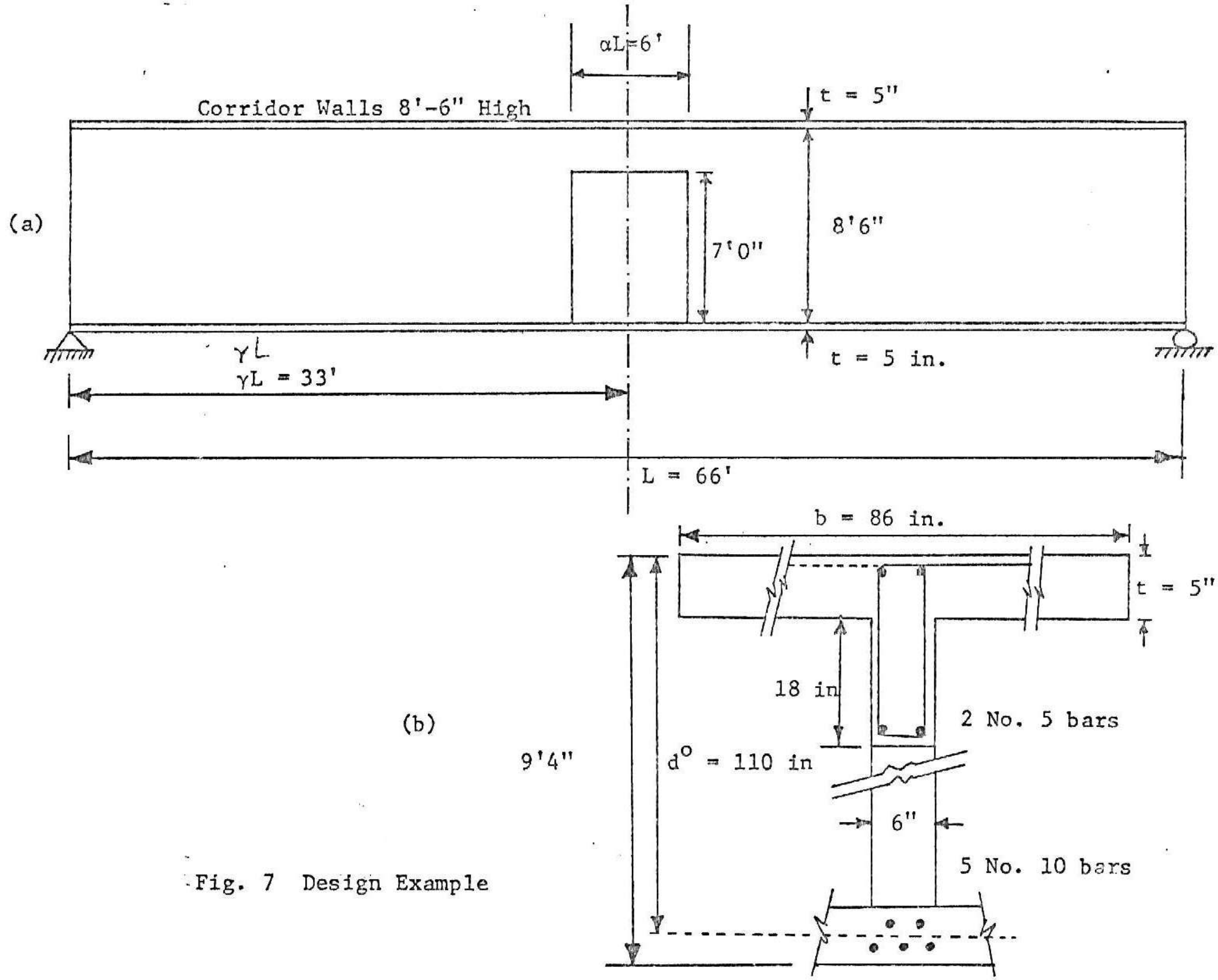


Fig. 7 Design Example

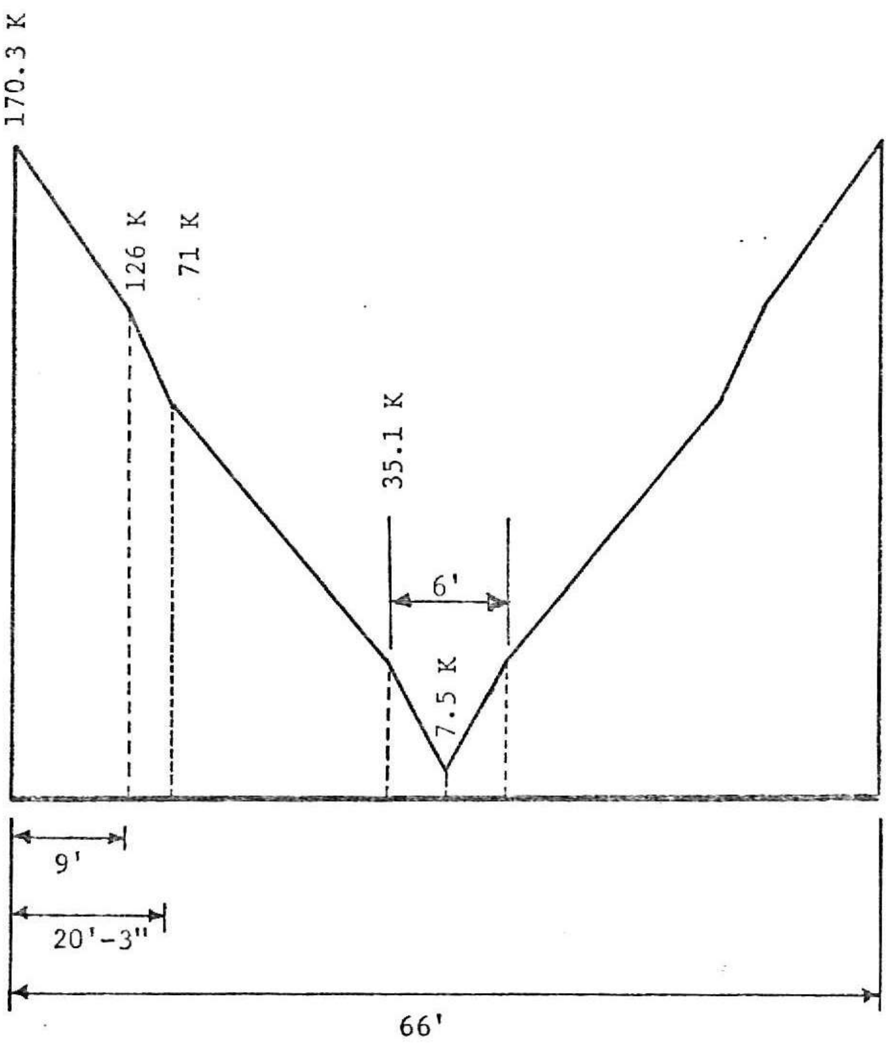
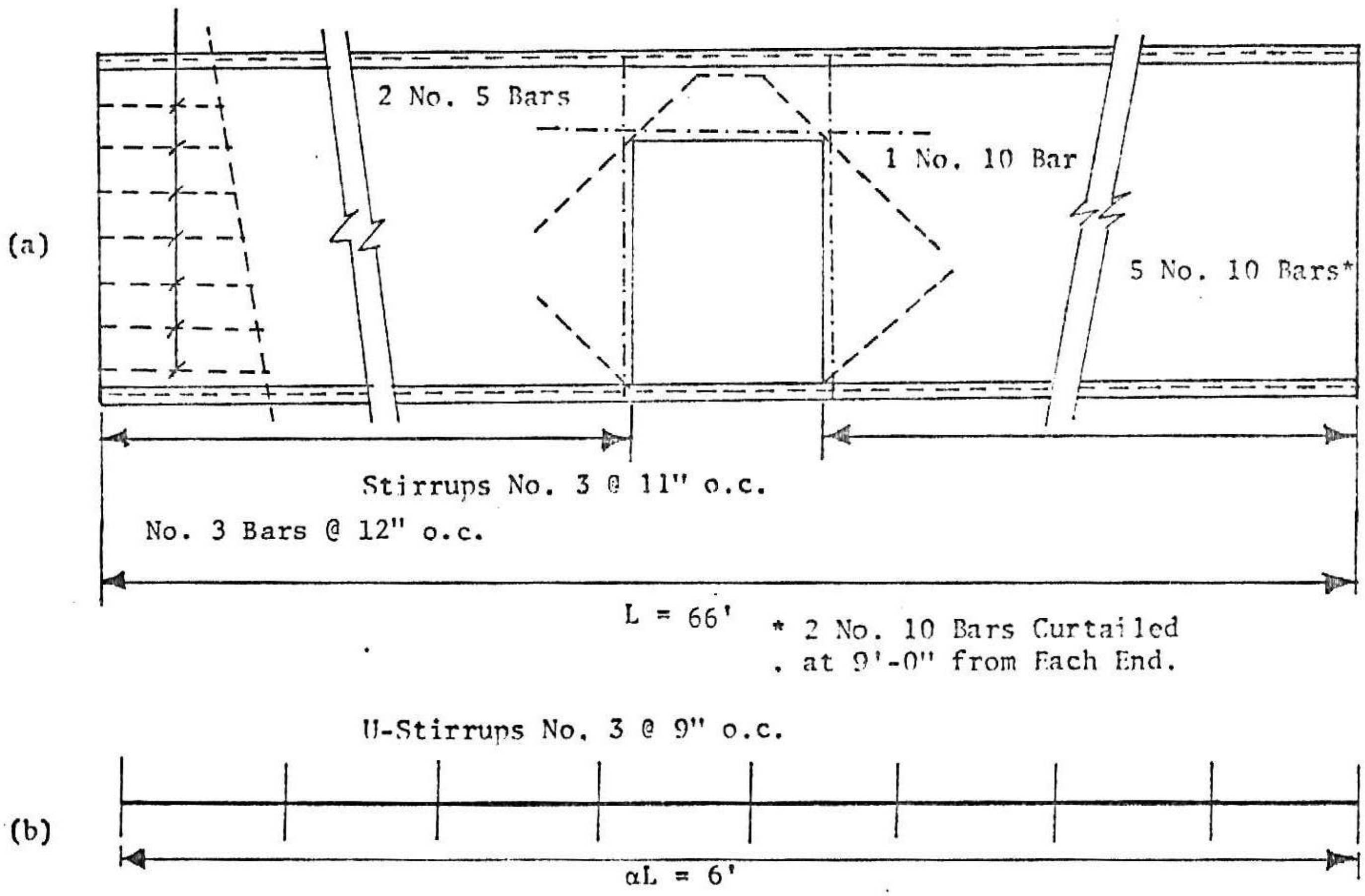


Fig. 8 Maximum Shear Envelope for Design Example



Spacing of Stirrups Over the Opening

Fig. 9 Reinforcement for the Wall-Beam

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ANALYSIS AND DESIGN OF TRANSVERSE, STAGGERED WALL-BEAMS

by

DILIP G. NAMJOSHI

B. E. (HONORS) Civil Engineering, 1969

S. P. College of Engineering, The University

of Bombay, India

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ABSTRACT

This report discusses a new concept in structural framing systems. Flat plate and shear wall type of construction is widely used for tall structures. The new system called 'Wall-Beam System' was the result of efforts made to find a way to resist loads more efficiently. The Portland Cement Association, Skokie, Illinois, conducted research to verify the adaptability of the wall-beam framing system. This research included an analysis and design of a high-rise building using wall-beams. Experimental investigation was also conducted on a half scale model of a wall beam.

A wall-beam is a reinforced concrete beam spanning across the entire width of a building. The wall-beam has one or more web openings for corridors, doors or windows and forms the principal element of this new framing system.

Structurally the wall-beam acts as an I-section cracked in tension. The floor slabs rest continuously on the wall beams. The locations of wall-beams are staggered on alternate floor levels. Thus, the floor slab is supported on two adjacent wall-beams on any floor level and suspended from the wall-beam on the floor above.

The web openings in the wall-beam tend to reduce the shear carrying capacity. Also stress concentration occurs around the corners of web openings.

Wall-beams have a large moment of inertia. At the lower floors the moment of inertia of supporting columns can be neglected and the wall-beams can be designed as simply supported beams.

In consideration of the usefulness of this new framing system in terms of architecture and planning, many advantages are observed. Planning flexibility is the important advantage. The structure can be designed using a basic module if required. Two story apartment units are easily accomodated in the wall-beam framing system.

This report gives the analysis and design of a wall-beam illustrated by a design example.