TILT UP CONCRETE WALL PANELS

by

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ABSTRACT

The design of tilt up concrete wall panels was studied in this paper. Design charts were developed to assist the engineer in the analysis of panels under load. Computer simulation of the concrete section was used to obtain the chart data.

A computer study of the effect of a number of parameters on the load carrying capacity of wall panels was made. Loading, section properties and support conditions were varied in isolation in an attempt to obtain general trends of how each influences the strength characteristic of a concrete wall panel.

A recommended design procedure has been included which utilizes the charts developed.

TILT UP CONCRETE WALL PANELS

TABLE OF CONTENTS

TABLE OF CONTENTS - Cont'd.

1. INTRODUCTION

Tilt up concrete wall panels have been employed in a variety of building types over the past several years. The recent introduction of a number of refinements in lifting hardware and techniques, combined with improvements in architectural finishes have allowed this type of construction to displace many of the more conventional approaches to building systems.

The most common application has been in load bearing walls for warehousing and light industrial structures. The standard 140 mm panel can be used for walls in excess of 8 meters in height supporting wind loads, roof loads and in-plane shear forces.

Because lateral deflections due to wind loading are frequently large, effects of slenderness must be considered. Design aids that are now available have been inadequate for the range of design conditions encountered and provide little guidance for situations falling outside their scope.

This paper deals primarily with stability considerations of load bearing panels spanning vertically between lateral supports. It is based on computer simulation of the concrete section under load and provides a rational basis on which to select the most suitable design. Every attempt was made to avoid the use of empirical formulas where extrapolation frequently leads to error.

2. THE ANALYTICAL MODEL

2.1 General Considerations

In the development of load capacity charts for concrete wall panels, it is important to have an accurate analytical model of the concrete section. Once curvatures are obtained for a given combination of axial compression and bending moment, the deflected shape of a panel under load can be computed.

Axial load is applied to the top of the panel at some eccentricity, in combination with a uniform lateral load. As the axial load is increased, deflections and moments also increase until no additional load can be sustained. The design charts are based on the peak value of axial load and the corresponding maximum applied moment.

2.2 Section Stiffness

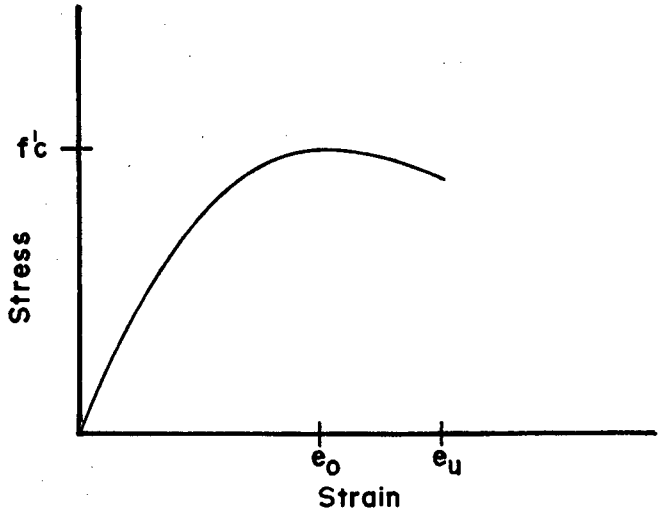
Moment-curvature relationships for the concrete section are obtained by numerical integration of the stress distribution for a given strain condition. The following assumptions have been made:

a. Concrete has a well defined stress strain curve which applies to both axial and flexural load, such as the one shown in Figure 2.1.
b. Reinforcing steel has a bilinear stress strain curve as also shown in Figure 2.1.
c. Sections originally plane and normal to the neutral surface remain so.
d. Perfect bond exists between steel and concrete.

FIGURE 2.1

STRESS-STRAIN PROPERTIES

CONCRETE



$$f_c/f'_c = (e/e_o)\,\frac{2}{1+(e/e_o)^2}$$

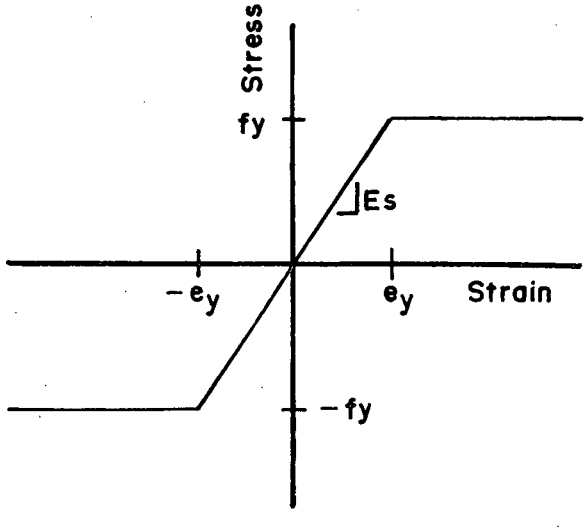
f'_c = 28 day comp. strength

$e_o = .002$ $\quad e_u = .003$

$f_c = 0.$ for $e > e_u$

or $e < 0.$

REINFORCING STEEL



$E_s = 200000$ MPa

$f_s = -f_y$ for $e_s \leq -e_y$

$f_s = E_s e_s$ for $-e_y < e_s < e_y$

$f_s = f_y$ for $e_s \geq e_y$

Following the assumptions that plane sections remain plane, the strain distribution can be defined in terms of two parameters; curvature and position of the neutral axis. Corresponding to each strain distribution, there is an equilibrium axial load and moment as shown in Figure 2.2.

By considering a great many such strain distributions, a moment-curvature relationship for a given axial load can be obtained by interpolation between data points.

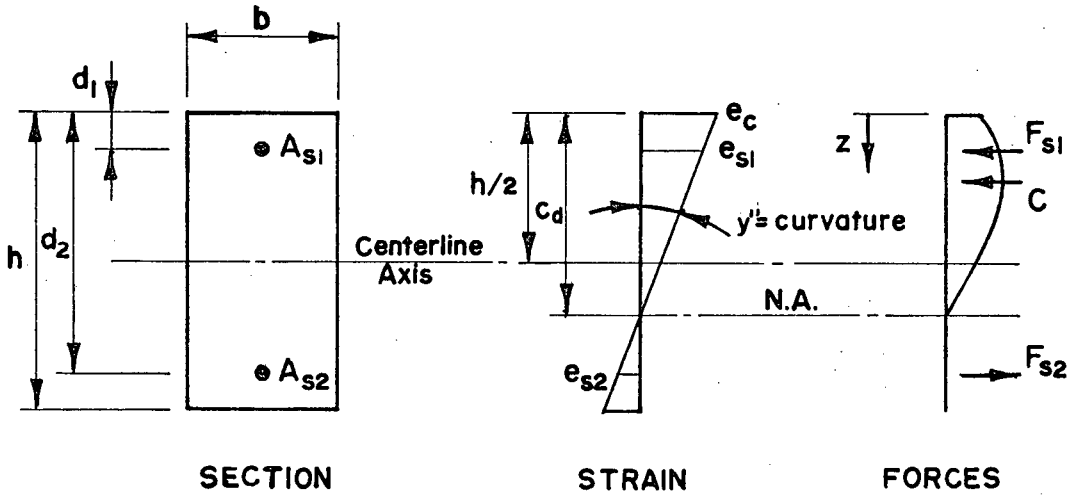
The technique involves a systematic generation of data points covering the applicable range of strain configurations. First of all, the curvature is held constant while the position of the neutral axis is varied in small increments from approximately 5% of the depth to twice the depth of the section. At each increment, the equilibrium axial load and moment are computed. Note that the moment is always calculated with respect to the centreline of the section. This is an arbitrary decision, but one that dictates that all external loads are applied at an eccentricity measured with respect to this centreline.

Next, the curvature is varied over a range that will encompass the entire interaction diagram. For each value of curvature, the neutral axis depths are varied and another line of constant curvature is obtained. A computer plot of typical curves is shown in Figure 2.3.

Once the load moment data has been generated for a desired section, it is a simple matter to find the moment-curvature relations. At any specified level of axial compression, the moment at each constant curvature line is obtained by direct interpolation. Figure 2.4 is a computer plot of several moment-curvature lines from the data of Figure 2.3.

FIGURE 2.2

DEVELOPMENT OF CONCRETE SECTION PROPERTIES



Strain Parameters :

$e_c = y'' c_d$ $\qquad$ $e_{s1} = y''(c_d - d_1)$ $\qquad$ $e_{s2} = y''(c_d - d_2)$

Concrete Stress $= f_c$
Steel Stress $= f_s$ $\Big\}$ From stress - strain relations

Force in concrete $\qquad C = \int b f_c \, dz$

Moment w.r.t. top $\qquad C_z = \int b f_c z \, dz$

$\bar{z} = C_z / C$

Force in steel $\qquad F_s = (f_s - f_c) A_s$

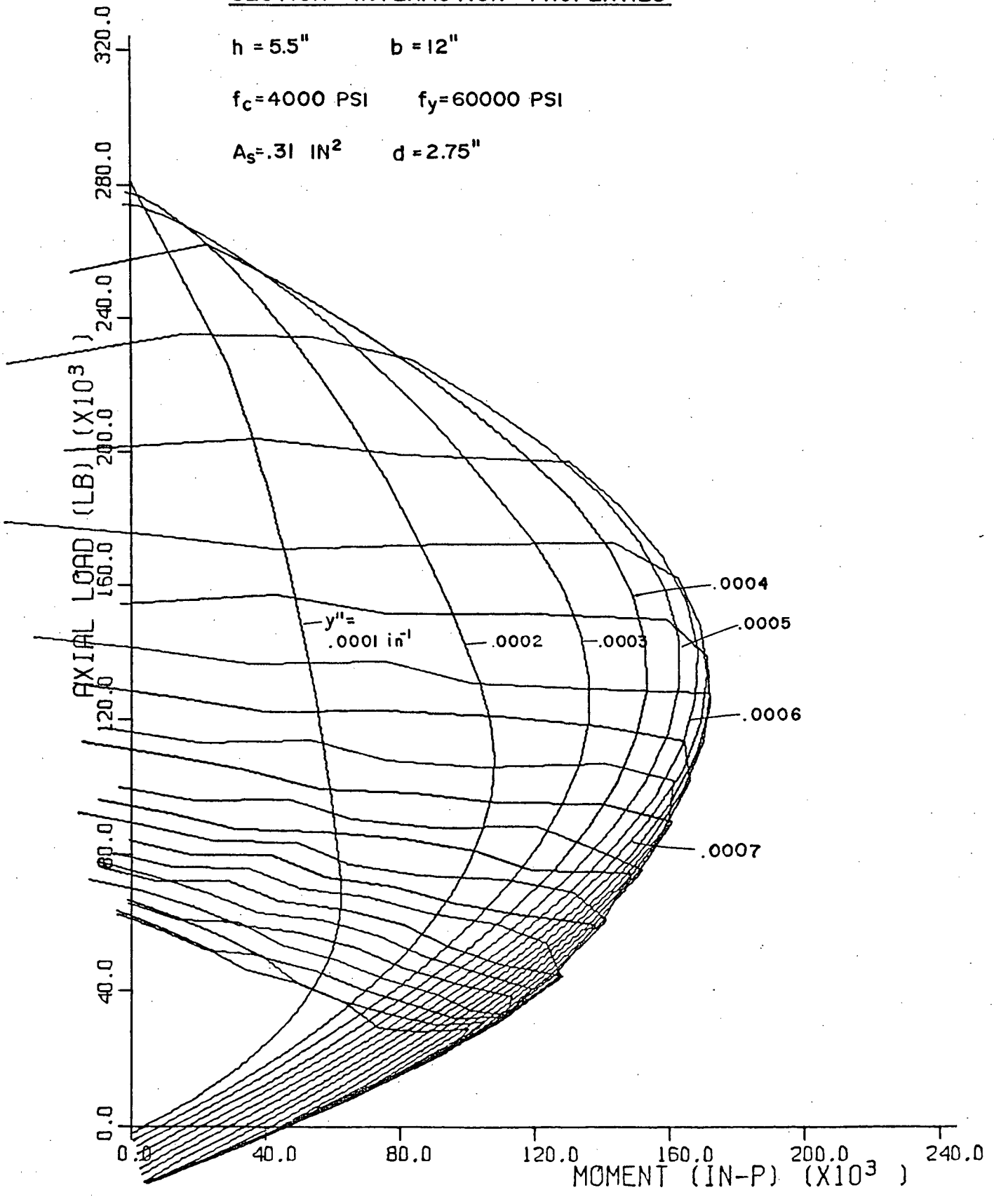
Total axial force $\qquad P = C + F_{s1} + F_{s2}$

Total moment w.r.t. centerline axis:

Moment $\quad M = C(h/2 - \bar{z}) + F_{s1}(h/2 - d_1) + F_{s2}(h/2 - d_2)$

FIGURE 2.3

SECTION INTERACTION PROPERTIES



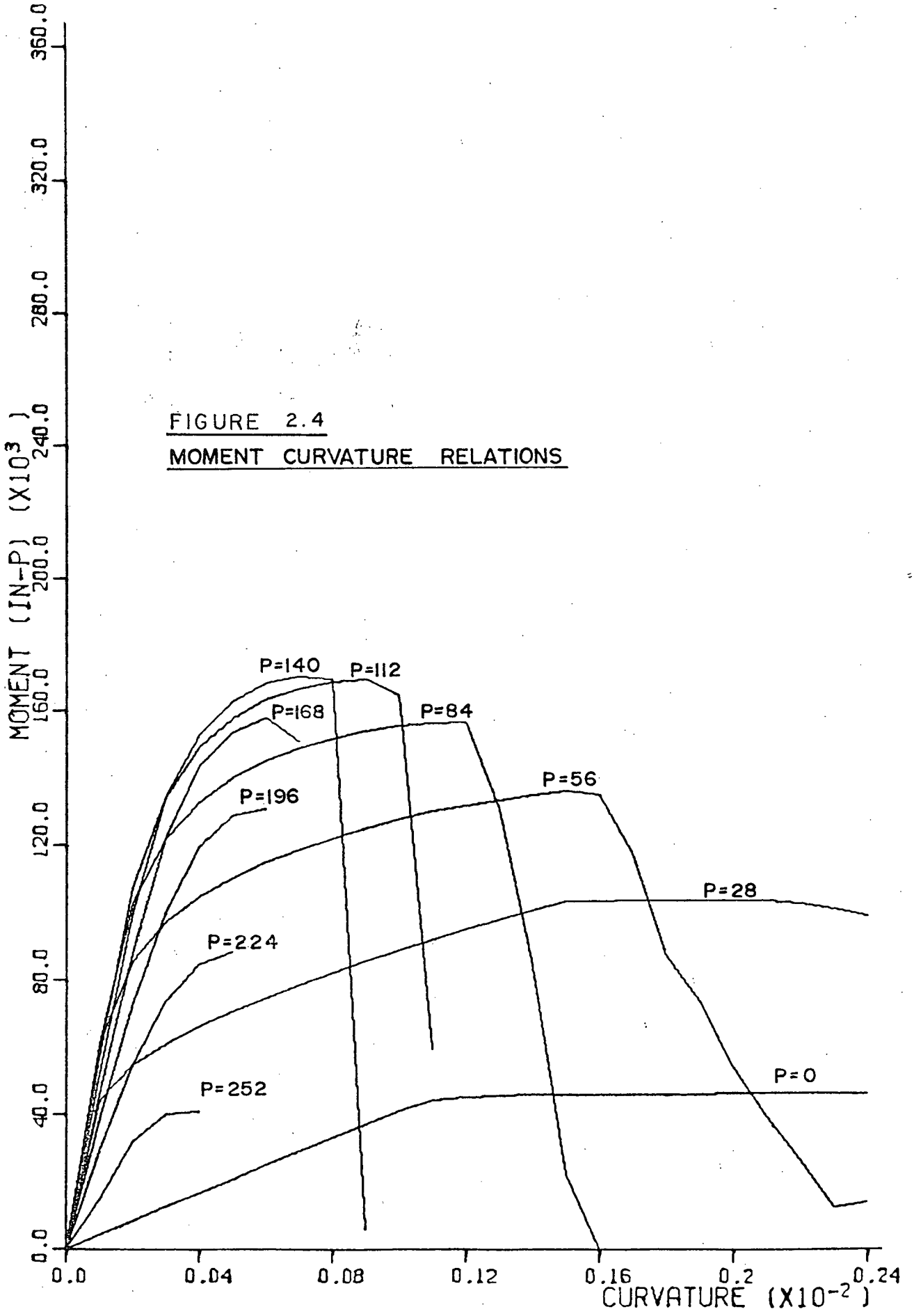


FIGURE 2.4

MOMENT CURVATURE RELATIONS

The desired curvature for a specified combination of axial load and moment is available by interpolation on the moment-curvature lines. Modern digital computers can readily generate and store this information for use in deflection calculations of a wall panel under load.

2.3 Wall Panel Analysis

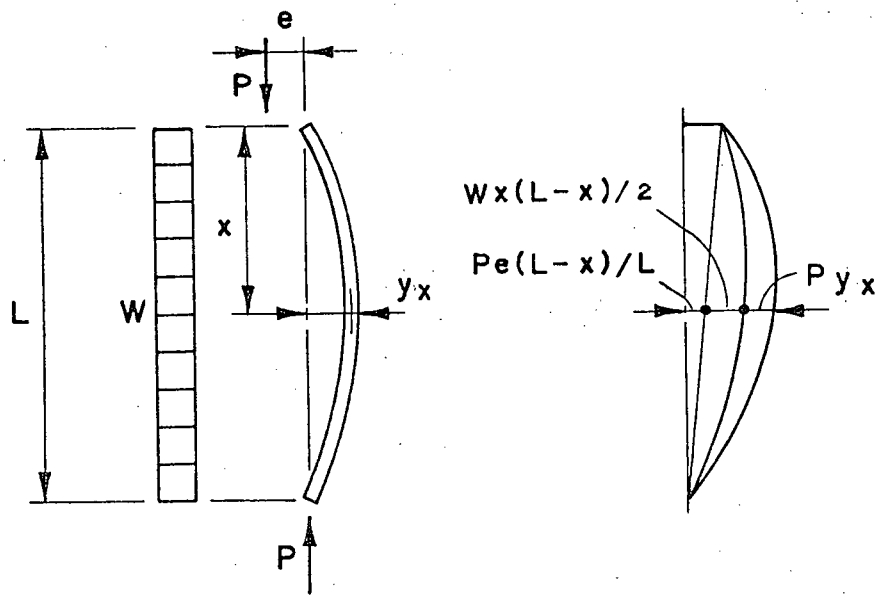
The purpose of the analysis is to obtain a relationship between applied loads and maximum bending moment in the wall panel. It is the magnitude of the maximum moment for a given axial/lateral load combination that is of interest, since it allows us to assess the available capacity remaining.

The loading configuration adopted, as shown in Figure 2.5, consists of a one dimensional beam (uniaxial bending) pinned at each end with a uniform lateral load along the entire length and an eccentric axial load at one (upper) end. From this model, it is possible to deduce the performance of many other design conditions.

A positive eccentricity is defined as that tending to increase the effect of lateral load and negative eccentricity as tending to decrease the lateral load effect. The maximum bending moment does not necessarily occur at mid-height nor does it necessarily coincide with the point of maximum lateral deflection. As previously mentioned, only the magnitude of the maximum moment is desired.

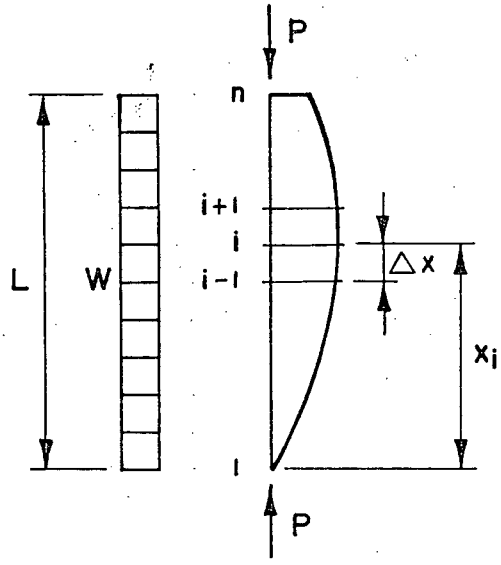
The procedure used to obtain load moment relations for a wall panel is shown pictorially in Figures 2.6 through 2.9. It is essentially a Newmark type method in which boundary conditions are assumed at one end and the equilibrium

FIGURE 2.5
CONCRETE WALL PANEL MODEL



DEFLECTED SHAPE BENDING MOMENT

FIGURE 2.6
COLUMN DEFLECTION CURVE



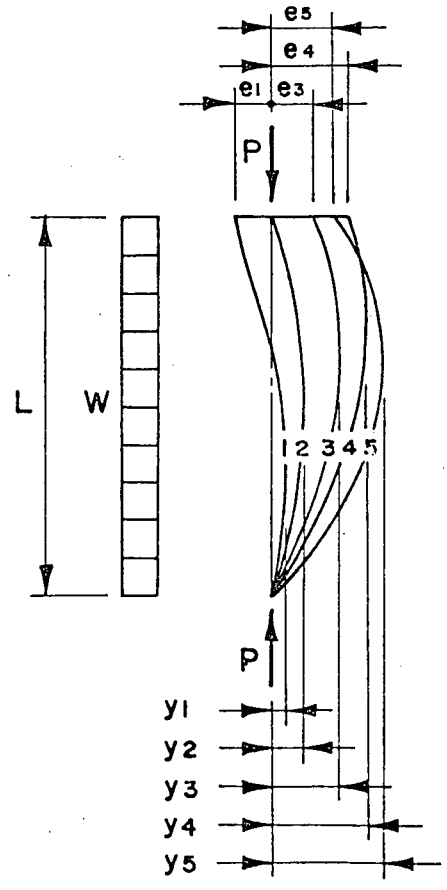
Moment = M
Deflection = y
Slope $y' = dy/dx$
Curvature $y'' = d^2y/dx^2$

$y_i = y_{i-1} - y'_{i-1}\Delta x + y''/2(\Delta x)^2$
$y'_i = y'_{i-1} + y''_{i-1}\Delta x$

$M_i = y_i P + M_{wi}$
$y''_i = f(M_i)$ from moment-curvature relations

M_{wi} = Moment due to lateral load

FIGURE 2.7
FAMILY OF C.D.C.'s



P = Constant
W = Constant

Initial slope at base is incremented

FIGURE 2.8

ECCENTRICITY MOMENT RELATIONS

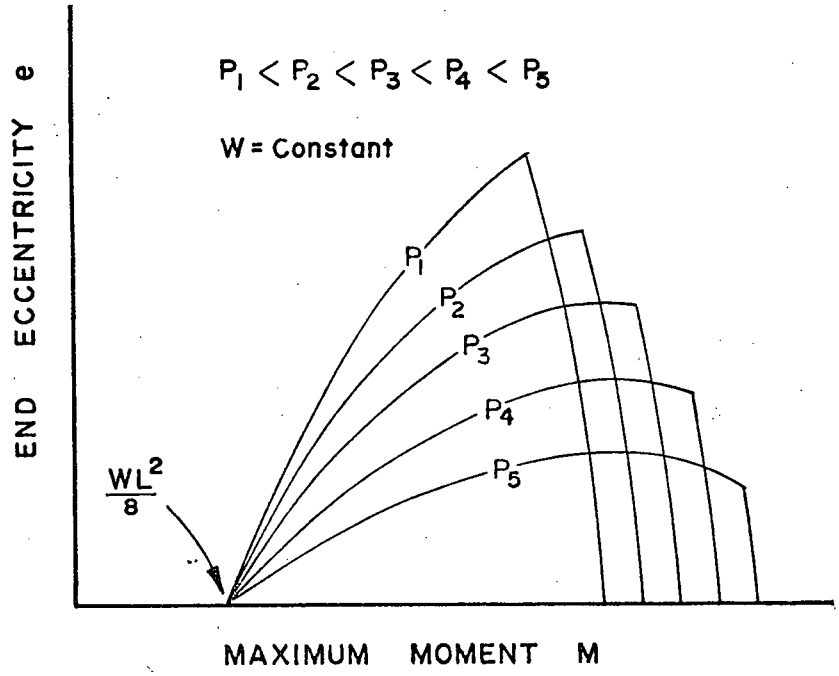
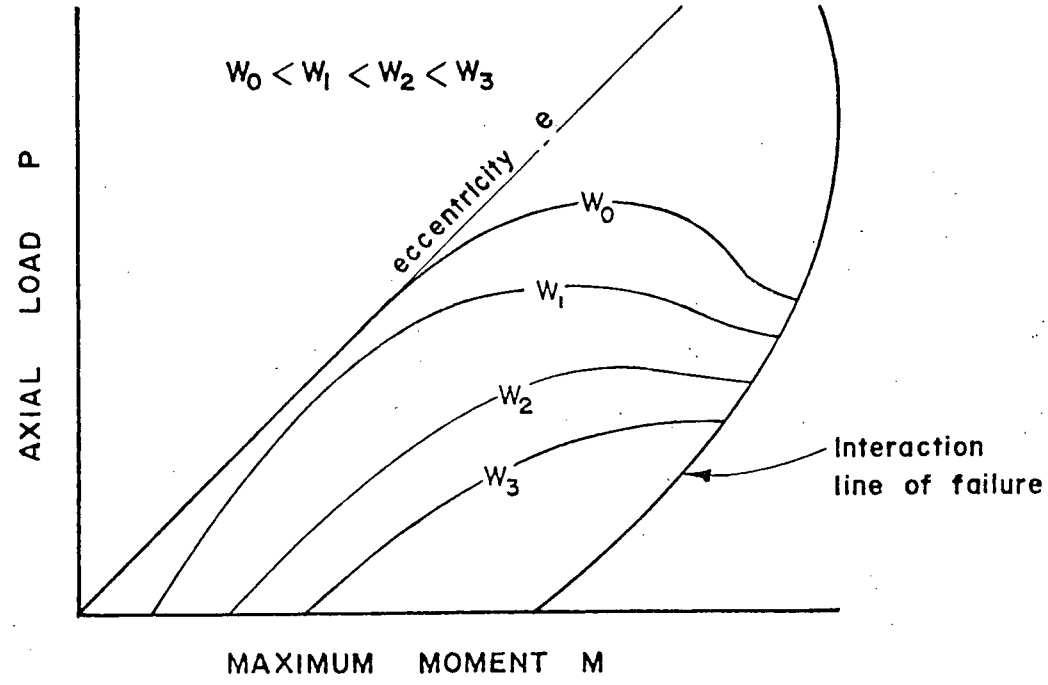


FIGURE 2.9

LOAD MOMENT RELATIONS



equation is integrated in segments along the member by making some assumption on the variation of curvature within the segment. Thus, the internal forces are computed for various deflected shapes and boundary conditions.

In Figure 2.6, the slope of the C.D.C. (column deflection curve) at the bottom is assumed and the resulting deflected shape computed at distinct nodes along the height. The resulting deflection at the top is the eccentricity required for equilibrium under that combination of lateral and axial load.

The starting slope is incremented as shown in Figure 2.7, until the end eccentricity decreases below some specified minimum value. In most cases, the end eccentricity will at first increase as the bottom slope is increased and then decrease to eventually become negative.

For a series of fixed axial loads, we obtain eccentricity-moment relations as in Figure 2.8. At any desired end eccentricity, the axial load P and corresponding maximum moment M are found by interpolation between the data points. The result is a load moment curve similar to that shown in Figure 2.9. Note that the axial load capacity is sometimes reached prior to actual material failure and the moment that occurs at this point is somewhat less than ultimate. The actual magnitude of this moment at or near the peak axial load capacity is very sensitive to slight variations in the loading conditions and material properties.

The maximum applied moment - that is the moment due to all applied loadings, but excluding P-delta magnifications - is well defined at peak axial load and is the basis on which the load capacity charts were developed.

2.4 Load Capacity Charts

The main considerations in the development of load capacity charts (see Appendix B) are that they should have the following properties:

a. Be simple to use and allow a comparison of alternate designs to be performed quickly and easily.
b. Accurately represent the performance of wall panels with a minimum number of design constraints.
c. Cover a broad range of design configurations to minimize the amount of interpolation or extrapolation.
d. Take adequate account of material variability and poor workmanship.

The layout of the charts was based on computations for the axial load capacity of a panel with a specific concrete section and fixed length. The peak value of load, applied at a positive end eccentricity, was found for various levels of lateral wind pressure.

Figure 2.10 shows the load moment curves for a panel subject to several levels of lateral load. The curves marked Wo, W_1, etc. show the value of the maximum magnified moment versus axial load for each lateral load W. The points marked B indicate the limit of useful capacity after which the descending load path is unstable. In curves W_3 and W_4, material failure occurred at points A before instability

FIGURE 2.10
LOAD MOMENT RELATIONS

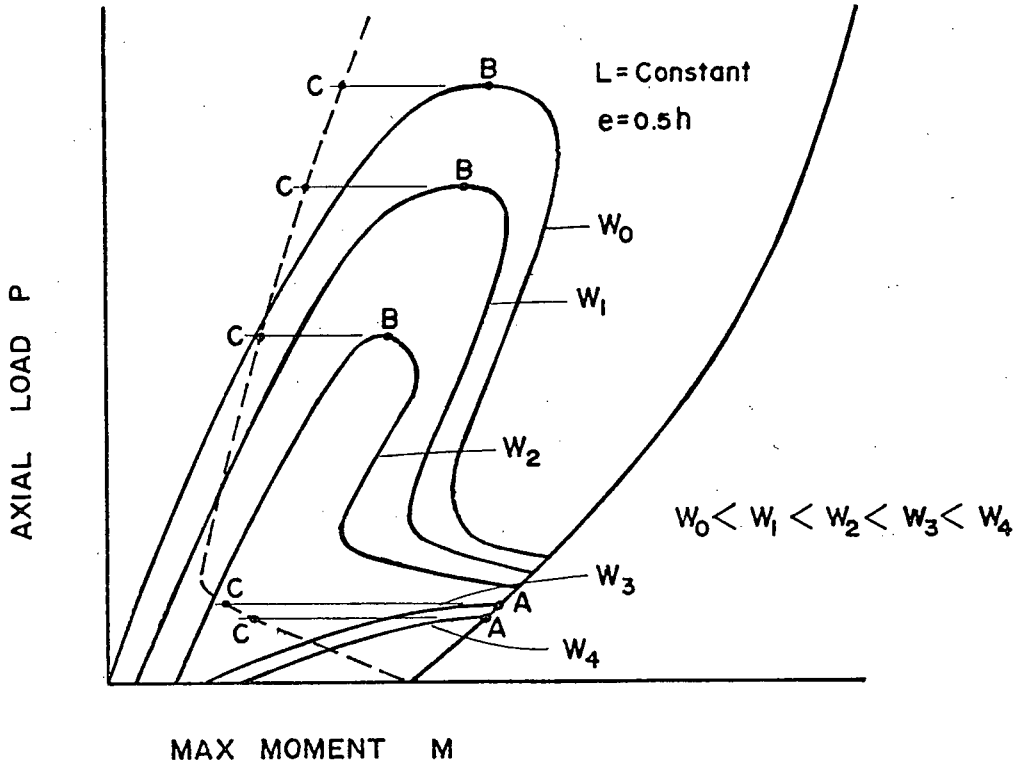


FIGURE 2.11
LOAD MOMENT CURVES

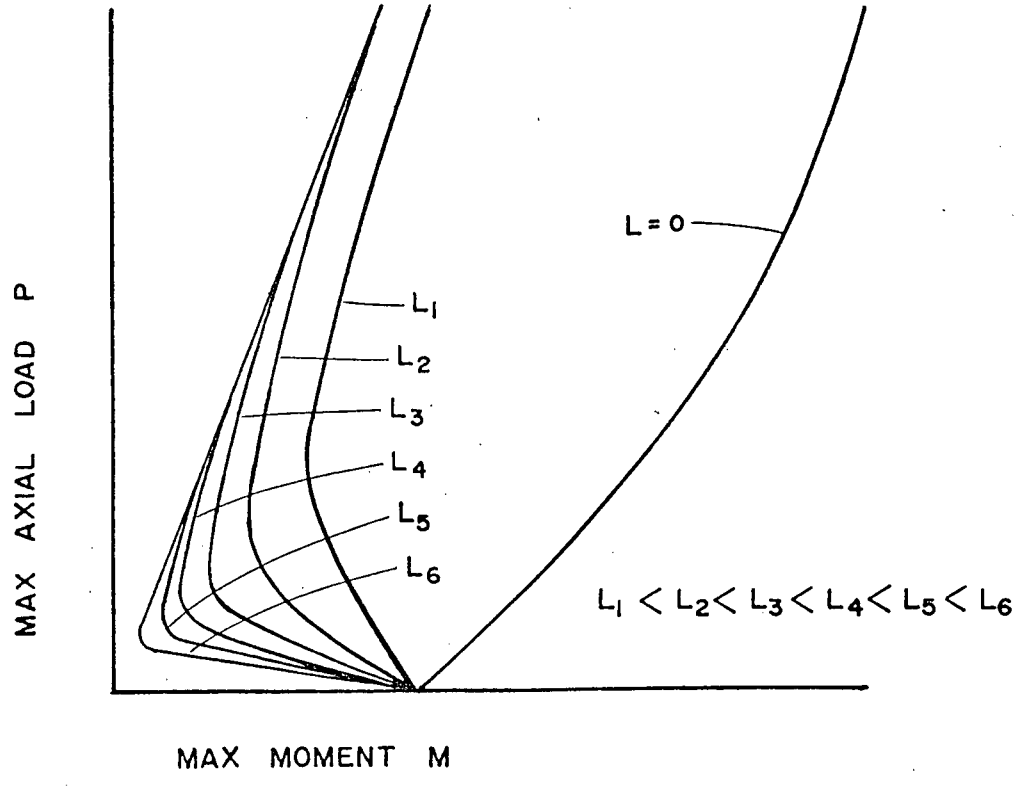
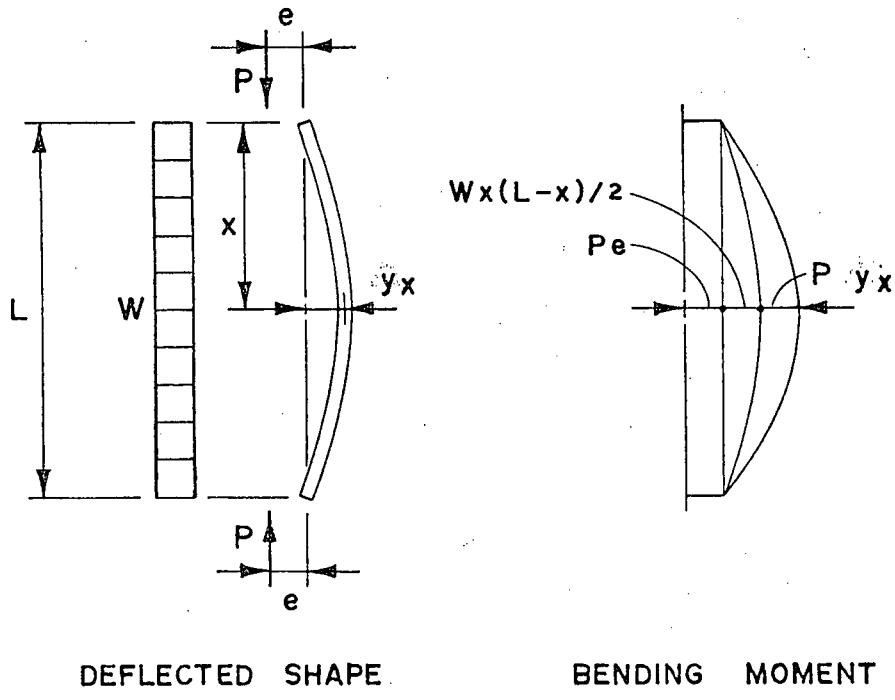


FIGURE 2.12
DOUBLE END ECCENTRICITY

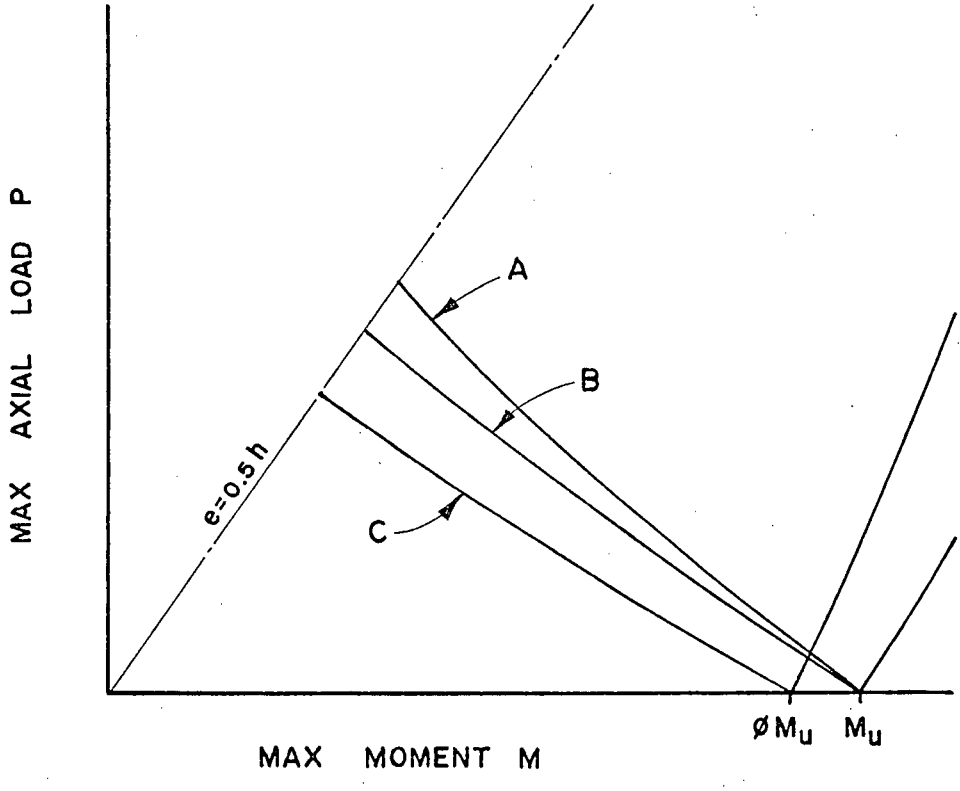


was reached. The dotted lines CC...C represents the maximum unmagnified moment corresponding to the points A or B. The designer enters the chart with the axial load and the maximum unmagnified moment; if this point lies to the left of CC...C, the design is safe. Other design curves for the same panel but different lengths are shown in Figure 2.11.

An end eccentricity of one-half the panel thickness was adopted as a cutoff point in the preparation of the charts. This is considered to be a minimum, particularly when axial load is large and lateral load moment is small. In most instances, however, the lateral load moment at or near mid-height of the panel is greater than the end moment and this restriction is of no consequence.

Figure 2.12 shows the deflected shapes and moment diagram of a panel with equal eccentricities at each end. The end conditions are such that the vertical load and the maximum unmagnified moment are the same as for the single eccentricity case in Figure 2.5. It will be seen that there is a small difference in the maximum magnified moments; the magnification being slightly less in the case of single eccentricity. In Figure 2.13, the curve A is obtained on the basis of single eccentricity, while curve B is based on double eccentricity. Curve C is the same as curve A with the capacity reduction factor included. It was felt that, in practice, single eccentricity is usually the case and the curves for the charts were obtained on that basis. That is to say, the vertical load applied at an eccentricity at the top equal to the half thickness of the panel is combined with the lateral load which would cause failure.

FIGURE 2.13
LOAD MOMENT CURVES



All the design charts of Appendix B include the capacity reduction factor as recommended by ACI/CSA codes. This means that the computed axial load and moment capacities are reduced by 0.9 to 0.7 depending on the level of applied axial load. In addition, the calculated section stiffness is reduced by the same capacity reduction factor. This has the effect of increasing deflections and lowering load capacities; it is, however, consistent with the requirements of the ACI/CSA codes. Curve C of Figure 2.13 illustrates the extent to which load capacity is decreased by application of the capacity reduction factor. This is discussed in greater detail in Section 3.6.

2.5 Constant Stiffness Method

Traditionally, hand calculations for deflection and stability of beam-columns assumed a constant value for the section stiffness EI along the member length, regardless of loading or degree of curvature. Although it is apparent that a constant EI for concrete covering all conditions is not correct, it is possible to obtain reasonably accurate results by assuming an average or effective value for this stiffness along the length of the member at a fixed level of axial load and maximum bending moment.

A good estimate of this desired value of average stiffness EI_{avg} is the secant modulus to the curvature-moment curve, or in effect, the inverse of the slope of a line connecting any point on the curve to the origin (see Figure 2.14). Typical values of EI_{avg} have been computed and plotted in Figure 2.15 for selected axial compressions. Also see Appendix C.

FIGURE 2.14

MOMENT CURVATURE LINES

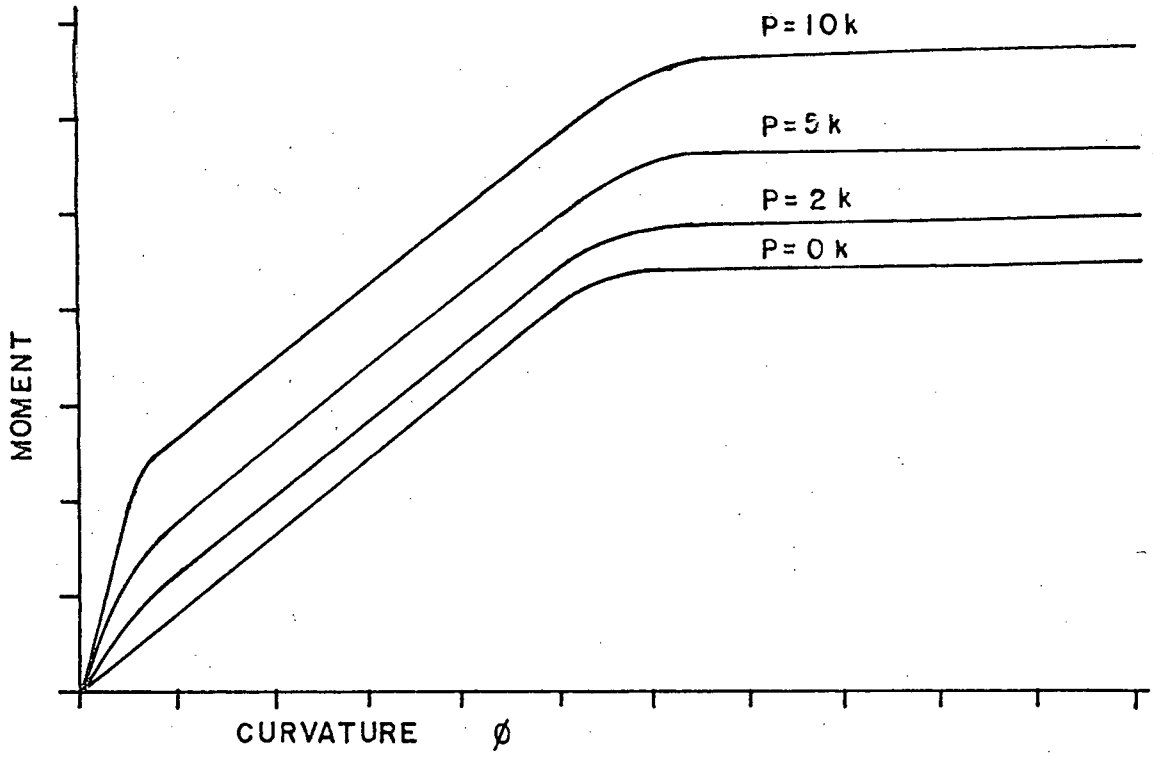
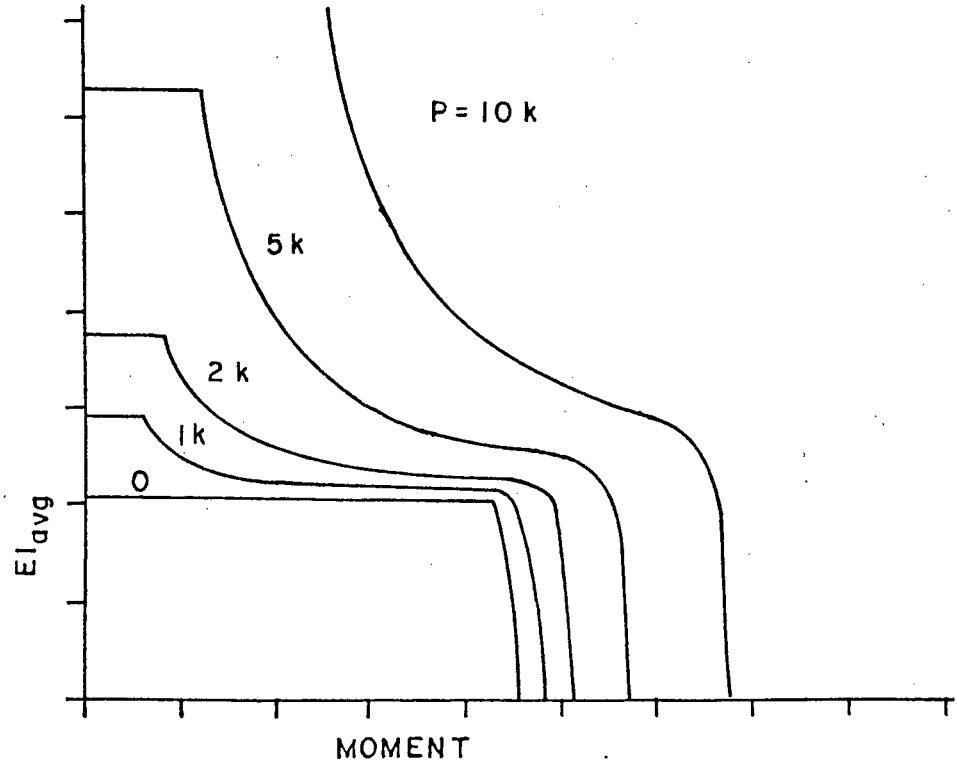


FIGURE 2.15

EI_{avg} CURVES



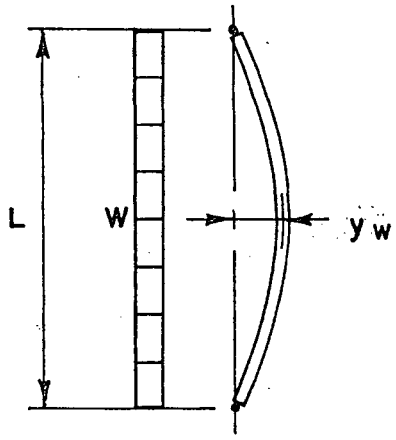
The procedure used to compute the maximum moment due to applied loading is as follows:

a. Set the effective height of panel (usually the clear distance between supports).
b. Compute axial load at the critical section (usually mid-height).
c. Compute the maximum moment Mo due to all applied loadings including wind load, end eccentricity and initial out of straightness (see Figures 2.16 through 2.18).
d. Estimate the final total moment; i.e. initial moment plus P-delta effects.
e. From the effective stiffness chart, find EI_{avg} for the given axial load and maximum moment.
f. Compute critical load P' and magnification factor (see Figure 2.19).
g. Compute total moment $M_t = M_o$
h. Check EI_{avg} and recompute M_t if necessary.

The effect of self-weight is approximated by assuming that one-half the panel weight acts at the top as concentric axial load (see Figure 2.20). Note that deflections at the top due to flexibility of the roof diaphragm do not contribute to the P-delta moment magnification unless there is some fixity at either support (see Figure 2.21).

This method of analysis gives results that compare favourably with a detailed computer analysis of the deflected shape. It does tend to be somewhat cumbersome for design, as a trial and error solution is required. Examination of the charts can, however, provide a useful means of evaluation of the effect of changes in material properties or section configurations.

FIGURE 2.16
LATERAL LOAD



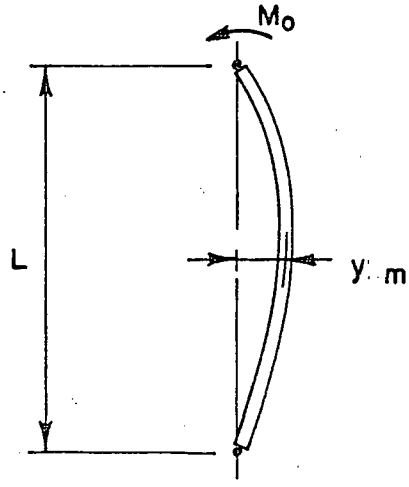
Maximum moment

$M_W = \frac{WL^2}{8EI}$ at $L/2$

Max. deflection

$y_W = \frac{5ML^2}{48EI}$

FIGURE 2.17
END MOMENT



Moment at midheight

$M_m = M_o/2$

Deflection at midheight

$y_m = \frac{M_o L^2}{16EI}$

For $y_w >> y_m$

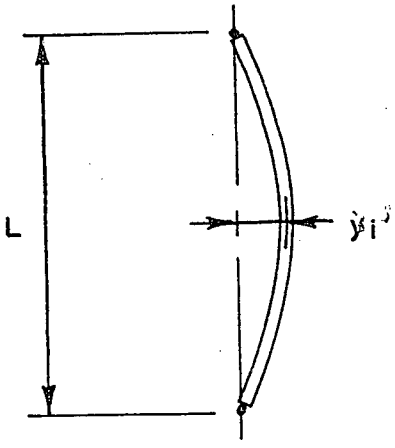
$y \approx \frac{ML^2}{\pi^2 EI} = \frac{M}{P'}$

$P' = \frac{\pi^2 EI}{L^2}$

M = max. moment

FIGURE 2.18

INITIAL DEFLECTION



Moment

$M = 0$

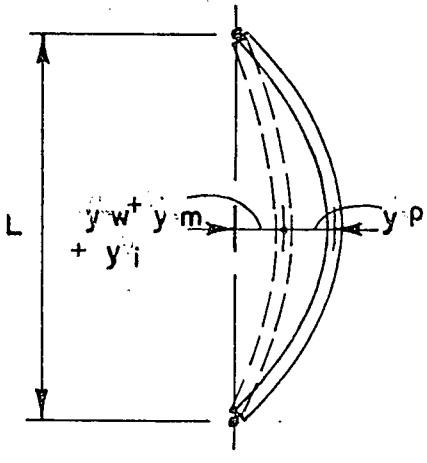
Deflection

y_i = Estimated

(initial out-of-straightness)

FIGURE 2.19

P-DELTA EFFECTS



Total deflection

$y_t = y_w + y_m + y_i + y_p$

$= M_t / P' + y_i$

y_p = additional deflection due to axial load

M_t = total moment

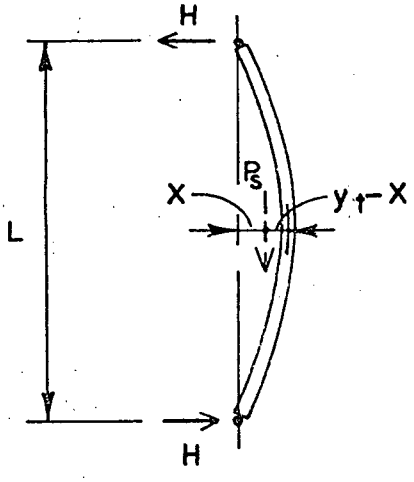
$= M_w + M_m + P\, y_t$

$$M_t = M_w + M_m + \frac{P}{P'} M_t + P\, y_i$$

$$= \frac{M_w + M_m + P\, y_i}{(1 + P/P')}$$

FIGURE 2.20

SELF WEIGHT



P_s = total weight of panel

y_t = total deflection due to all effects

X = C. of G. of panel

H = horiz. reaction

$= P_s X/L$

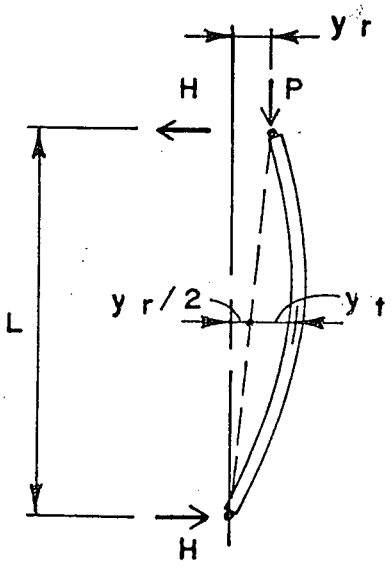
Moment at midheight

$M = HL/2 + \frac{P_s}{2}(y_t - X)$

$= P_s y_t/2$

FIGURE 2.21

ROOF DEFLECTION



y_r = roof deflection

$H = P y_r/L$

Moment at midheight

$M = HL/2 + P(y_t - y_r/2)$

$= P y_t$

3. DESCRIPTION OF INVESTIGATIONS

3.1 General

The behavior of concrete wall panels has been investigated for a wide variety of loading conditions and material properties. The computer programs used for this purpose are capable of predicting the curvatures and deflections to a high degree of accuracy with a minimum number of simplifying assumptions.

The big advantage in computerized analytical methods over actual prototype testing, other than the cost factor, is the ability to look at the effect of varying isolated parameters while the remainder are held constant.

In the following sections, a parametric study is made of the effects of varying loading conditions, material properties and panel dimensions. A standard configuration was adopted as a basis of comparison which represents an average practical case: 5½" panel thickness, 20' unsupported height, pinned supports, 4" axial load eccentricity, 30 PSF lateral load.

3.2 Loading

The effect of the inter-relation between axial and lateral loads on the structural performance of thin wall panels is the main purpose of these investigations. Lateral loads usually take the form of wind pressures that are uniformly distributed along the height, but will sometimes take the form of a series of point forces exerted by window or door frames. Overhanging features such as fascia framing may also provide a source of lateral loading usually resulting in a couple.

Axial loads are vertical forces applied at the top of the panel and sometimes at some location along the height. External loads such as reactions from roof or floor members are usually applied eccentrically with respect to the centreline of the cross section. Self-weight contributes a significant portion of the axial load and is considered to be concentrically applied. Additional axial loads may also occur as a result of in-plane forces.

For the common tilt-up application where unsupported heights exceed 18 feet and roof loads are in the order of 1000 to 1500 PLF, the contribution made by wind pressure is easily the largest influence on final bending moments in the wall panel. With the added effects of axial load and slenderness magnifications of moment, the expected mode of failure would be one of instability. Conversely, for short panels of 10 to 12 foot heights, the effect of lateral wind load is considerably less and failure would likely occur as a result of excessive bending at the eccentric support, particularly when large axial loads are applied.

The charts of Appendix D give a good general indication of how the capacity of a panel is affected by changes in axial and lateral load. Linear interpolation between the plotted parameters will give accuracies sufficient for design purposes. If required, the curves may be extrapolated beyond the lateral loads given by simply extending the given lines to the intersection of the zero axial load line. The value of a lateral load at this point is given by

$$W = \frac{8Muo}{L^2}$$

where Muo is the ultimate moment at zero axial load

L is the distance between supports

The effect of self-weight was not included in the charts of Appendix D but can be approximately accounted for by assuming that one-half of the weight is applied (concentrically) at the top. This has been verified by analyzing a panel with actual self-weight and then with zero weight but an equivalent axial load of one-half the weight of the panel at the top. The results are virtually identical.

It might appear more accurate to apply as vertical load at the top, the weight of panel above the point of maximum moment, but this is almost always about equal to one-half the panel weight where slenderness magnifications become significant.

3.3 End Eccentricity

Variations in end eccentricity at the top of a panel have a substantial effect on the load carrying capacity as can be seen by the two charts of Figure 3.1.

For small eccentricities the peak load capacity is high, with a substantial range of unstable equilibrium before actual material failure. If the axial loads are allowed to approach the indicated maximum, there exists the danger of "snap through buckling" in which the panel may jump from the stable to unstable portion of the curve. This may be initiated by unexpected external effects like sudden impact or high wind gusts. The consequence would be a sudden and total loss of load capacity.

This phenomenon essentially does not occur when eccentricities are at least h/2 and the lateral loads exceed 10 PSF (0.5 kN/m^2). For these conditions, the load moment curve increases steadily to a maximum just prior to ultimate moment capacity.

FIGURE 3.1a

TYPICAL LOAD MOMENT CURVES

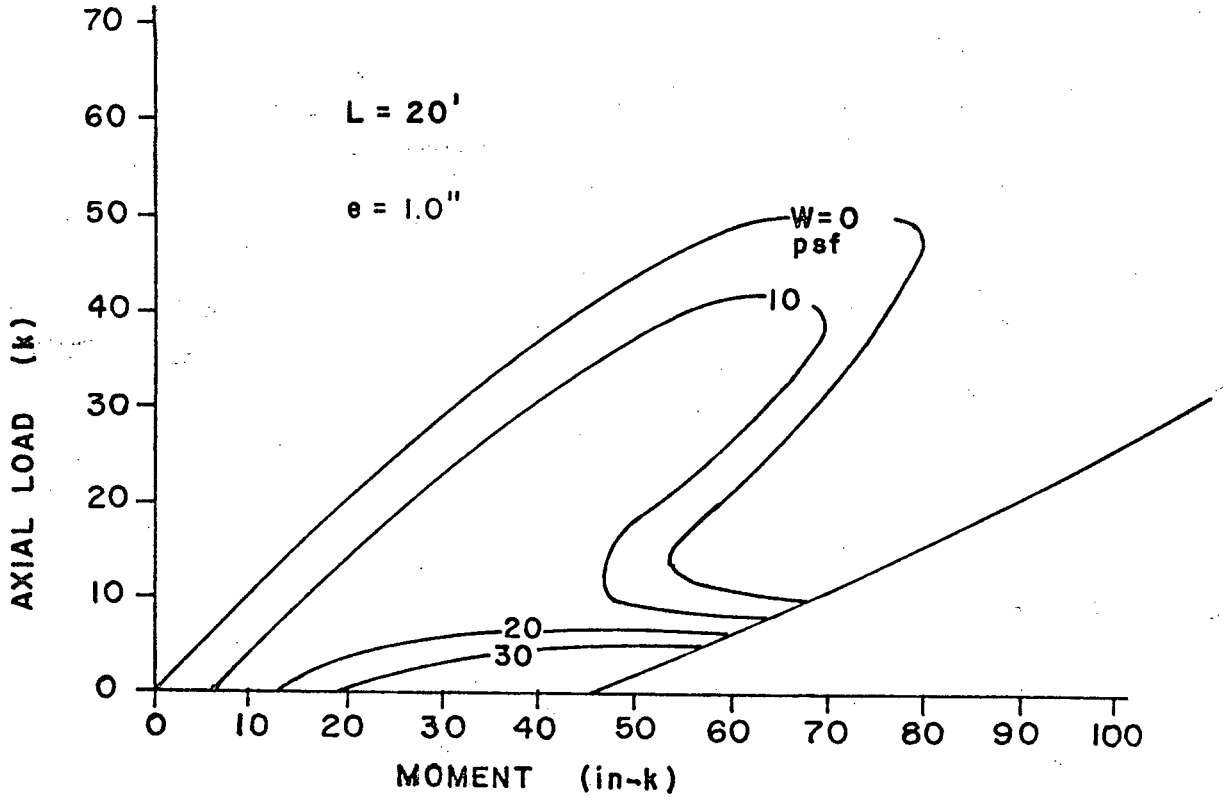
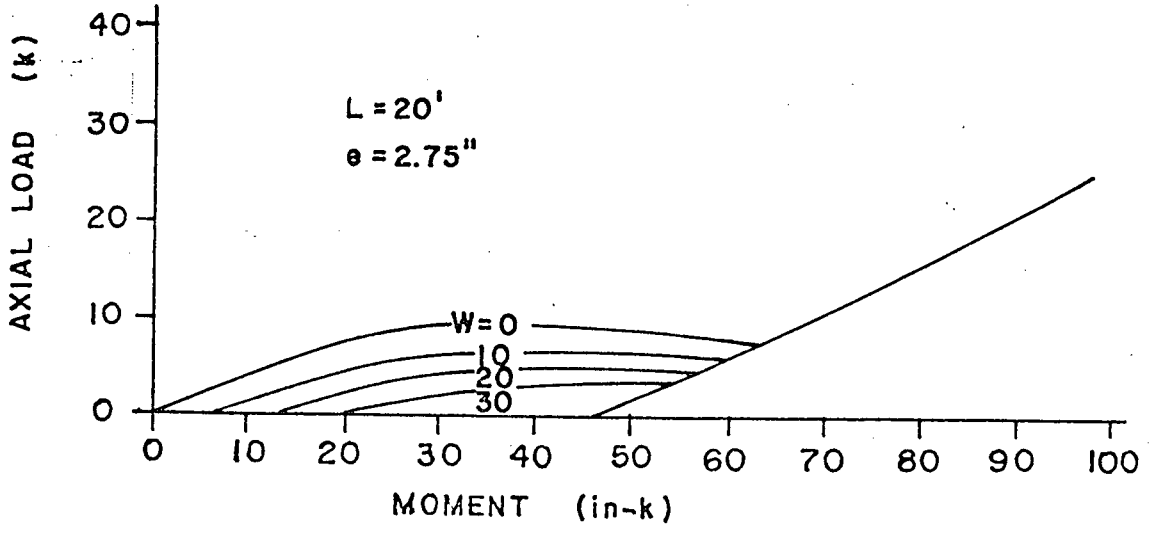


FIGURE 3.1b

LOAD MOMENT CURVES



This situation is more predictable and the dangers in collapse can be adequately guarded against by careful selection of loadings and capacity reduction factors.

Negative eccentricities produce a double curvature condition resulting in lower deflections and higher load capacities. Failure is often the result of excessive bending moment at the eccentric support and can be predicted by a simple strength calculation. For the case where the failure mode is one of instability due to positive bending (high panels with full wind load and small negative end eccentricities), there is usually an abrupt loss of load capacity well before material failure.

3.4 Concrete Properties

For the purpose of these investigations, the concrete stress strain relation was assumed to take the shape shown in Figure 2.1. It is considered that this is a sufficiently realistic representation of the actual curve.

Compressive Strength

For all design charts of Appendix B, the 28 day compressive strength was assumed to be 25 MPa (3620 PSI). This is normally a minimum requirement for adequate resistance against flexural cracking during the panel lifting operation. The effect of varying concrete strength on the average stiffness properties is shown for one specific case in Figure 3.2(a). For the most part, the ultimate moment capacity is not greatly affected by changes in concrete compressive strength. The stiffness does seem to increase with increasing compressive strength up to about 4000 PSI. After this, there is very little increase.

FIGURE 3.2a

AVERAGE STIFFNESS FOR

VARIATIONS IN CONCRETE STRENGTH

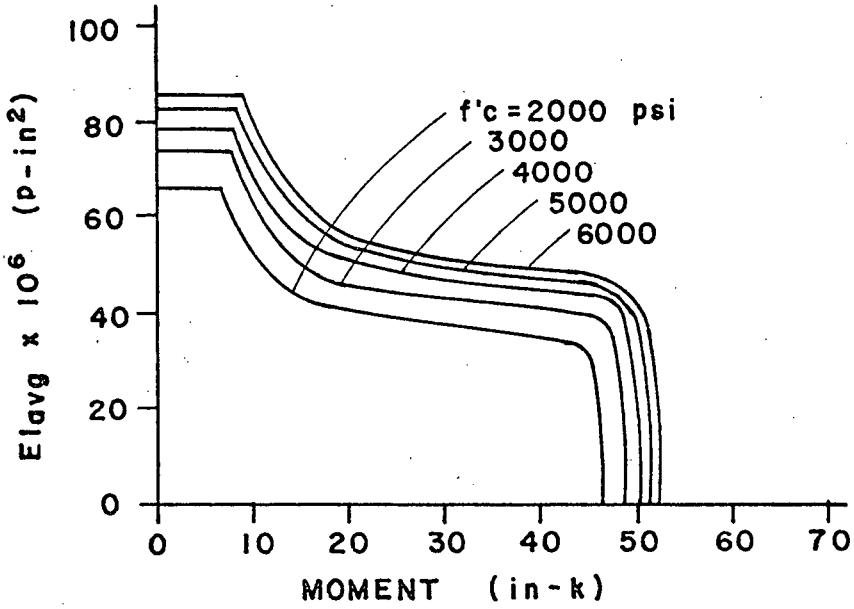
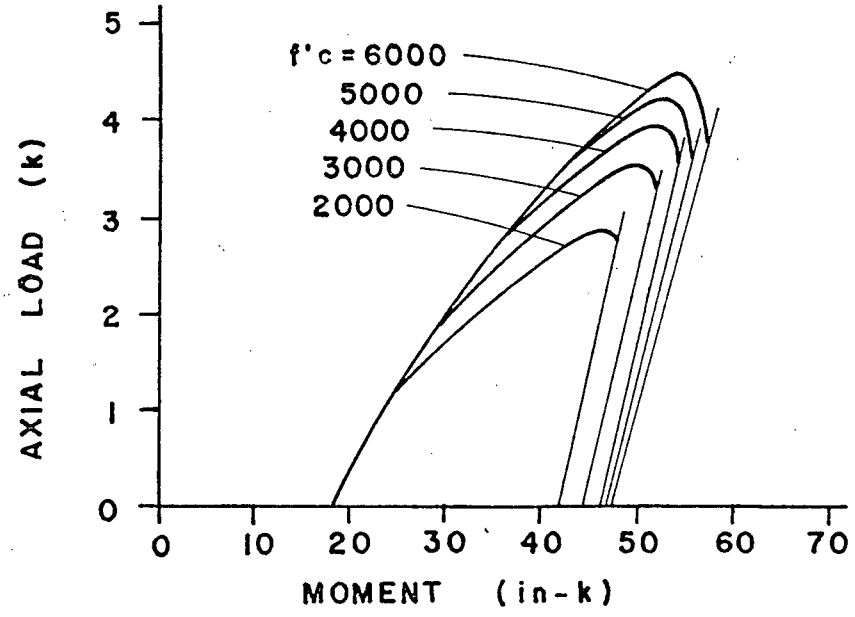


FIGURE 3.2b

LOAD MOMENT RELATIONS FOR

VARIATIONS IN CONCRETE STRENGTH



The effect of compressive strength changes on load capacity is shown in Figure 3.2(b). A decrease from 4000 PSI to 2000 PSI reduces the peak axial load by about 25% and an increase from 4000 PSI to 6000 PSI increases this axial load by 15%.

Tensile Strength

Consideration of concrete tensile strength will add to the overall stiffness of the section at low bending moment but will not result in any significant increases at ultimate.

A comparison of the average stiffness for 0, 300 PSI and 600 PSI ultimate tensile capacity is shown in Figure 3.3. A check on axial load capacity for the same three cases revealed no differences, presumably because the stiffnesses are indistinguishable at higher moments.

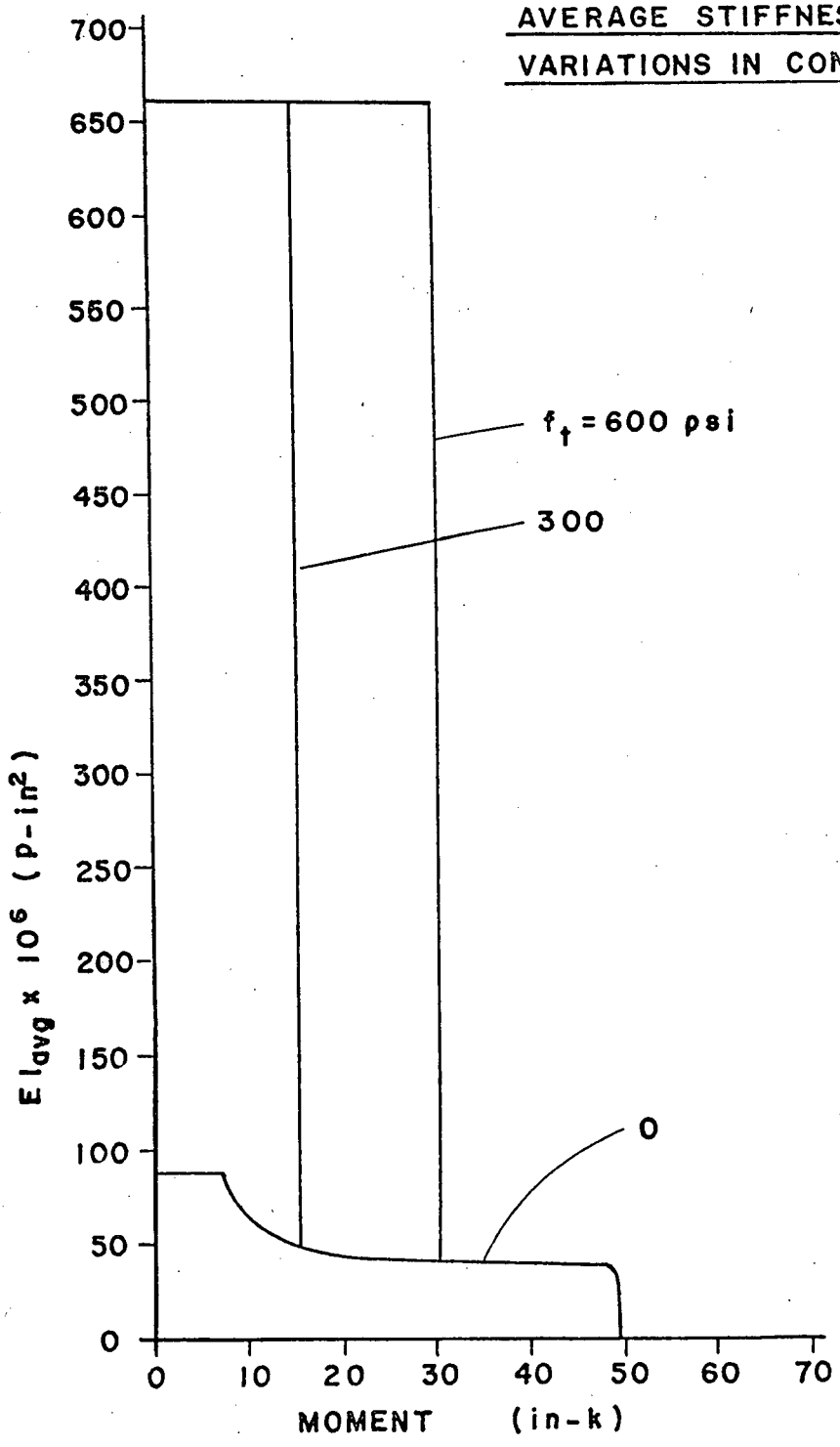
3.5 Reinforcing Steel

A bi-linear elasto-plastic stress strain curve was adopted for the reinforcing steel as shown in Figure 2.1. The steel is assumed to elongate indefinitely at a constant stress after reaching the specified yield.

Perfect bond between concrete and steel is assumed such that strain in the steel is always proportional to curvature. This implies a uniform distribution of cracks on the tension face with no slippage at the cracks. The assumption is generally believed to be valid for deformed reinforcing bars of yield strength up to about 60 ksi.

FIGURE 3.3

AVERAGE STIFFNESS FOR VARIATIONS IN CONCRETE TENSION



Yield Stress

Although the steel yield stress for all the investigations was set at 60 KSI, the effect of lower or higher yields was investigated. Figure 3.4(a) shows the stiffness variations for 40, 60 and 80 KSI yield and for the fictitious case of infinite yield strength. The ultimate moment capacity increases with increasing yield, as would be expected at low levels of axial compression, but the average stiffness does not change.

A comparison of axial load capacity for various yields in reinforcement is shown in Figure 3.4(b). The increase in load capacity can be attributed to the additional ultimate moment at higher yields.

Because of the variations in these results due to axial compression, there does not appear to be any satisfactory relation available for modelling the effect of lower or higher yield stress. However, for the normal range of axial loadings for tilt-up wall panels (0 to 2000 PLF), axial load capacity appears to vary in roughly a linear fashion with yield stress of the steel.

Area of Reinforcement

Variations in the areas of steel used in a section appear to affect the section stiffness on an approximately linear basis. The charts of Appendix C indicate that the assumption of a linear variation would be sufficiently accurate for changes of the order of ±15%.

The axial load capacity is also affected by changes in areas of steel and again, the relation is almost linear (see Appendix B).

FIGURE 3.4 a

AVERAGE STIFFNESS FOR VARIATIONS IN REINFORCING STEEL YIELD STRESS

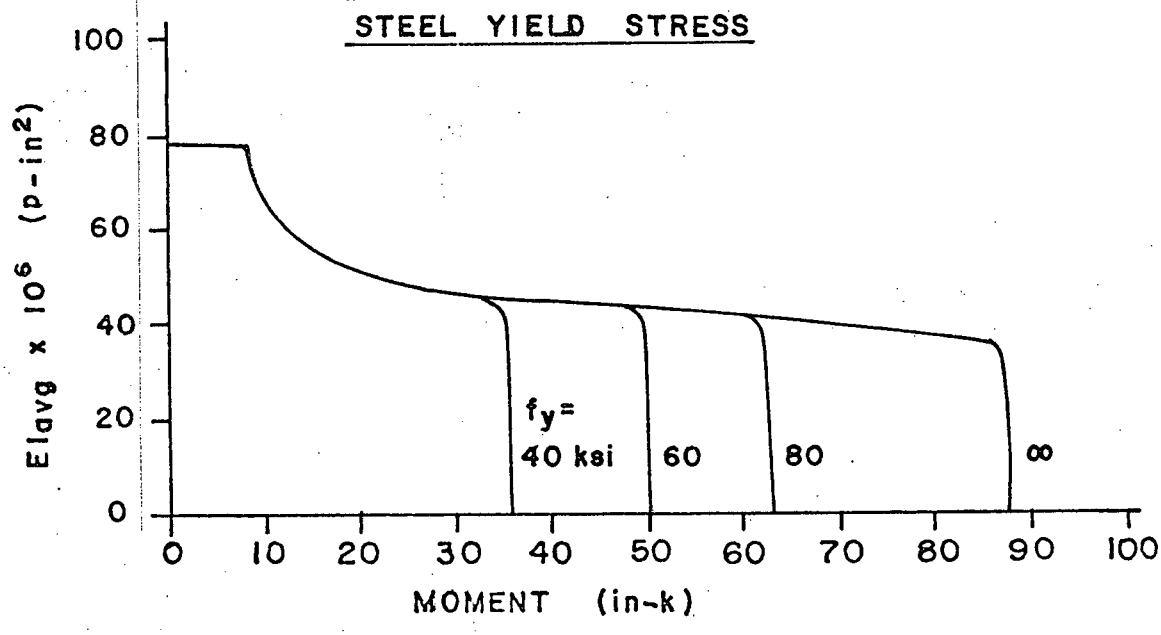
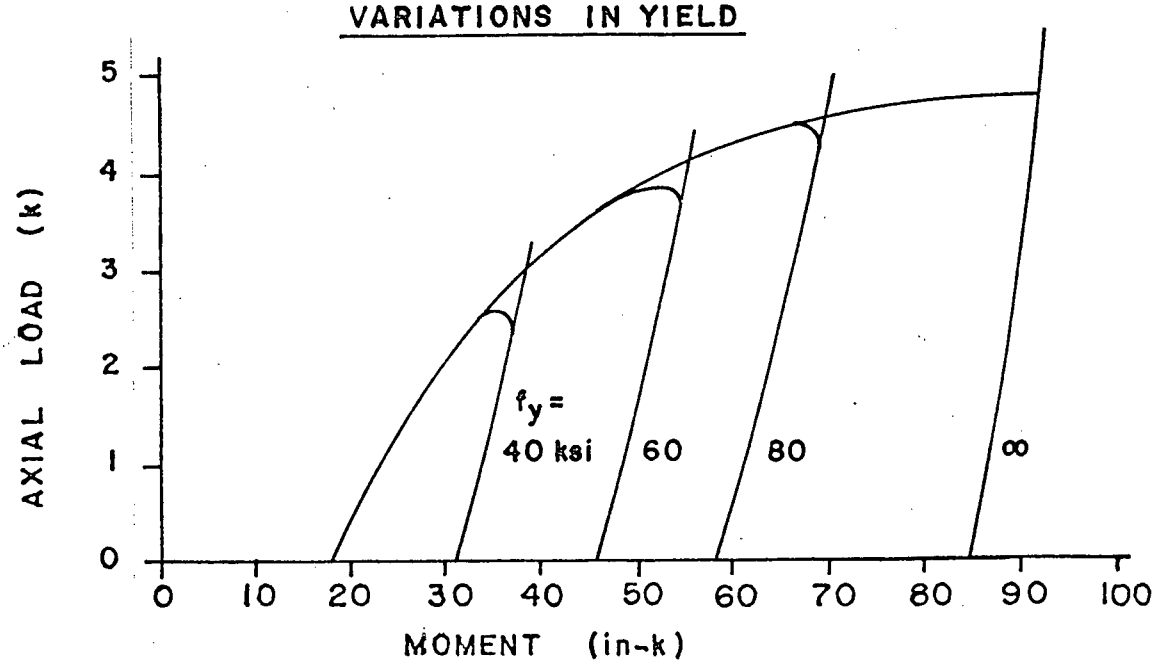


FIGURE 3.4 b

AXIAL LOAD FOR VARIATIONS IN YIELD



Location of Reinforcement

Location of reinforcing steel in the section has a pronounced effect on the load carrying capacity of the panel. For a 5½" section with only one layer of reinforcement, variations in placement of ±1/8" caused changes in stiffness of about 10% and in peak axial load by 13% (see Figure 3.5).

A panel with two layers of steel (Figure 3.6) showed stiffness variations of 4% and load variations of 7% for the same 1/8" tolerance. In both cases, the consequence of small changes in location of the steel appear to be approximately proportional to at least the square of the effective depth of steel.

Prestressing

Prestressing of the reinforcing steel can result in large increases in bending stiffness, particularly for low levels of axial loading to which most wall panels are normally subjected. A comparison between a prestressed and non-prestressed wall panel is shown in Figure 3.7.

Prestressing allows for full advantage to be taken of the increased stiffness at higher axial compression without the external instability due to P-delta effects. Since this paper is concerned with the non-prestressed condition, no further discussion of prestressing will be made.

3.6 Capacity Reduction Factor

The capacity reduction factor Ø as recommended by the ACI/CSA codes (ref. 2 and 3) should be used to decrease the computed load capacities. The purpose of this reduction is to allow for variations in material properties and poor workmanship.

FIGURE 3.5 a
AVERAGE STIFFNESS FOR VARIATIONS IN REINFORCING STEEL LOCATION

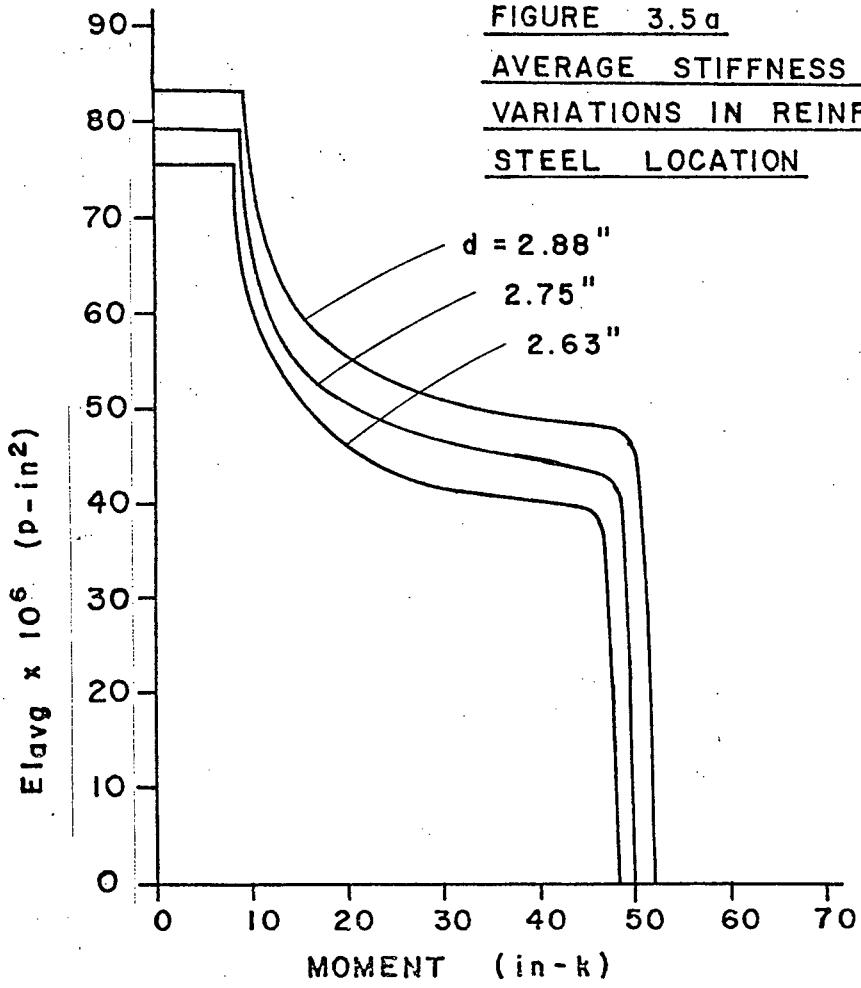


FIGURE 3.5 b AXIAL LOAD FOR VARIATIONS IN STEEL LOCATION

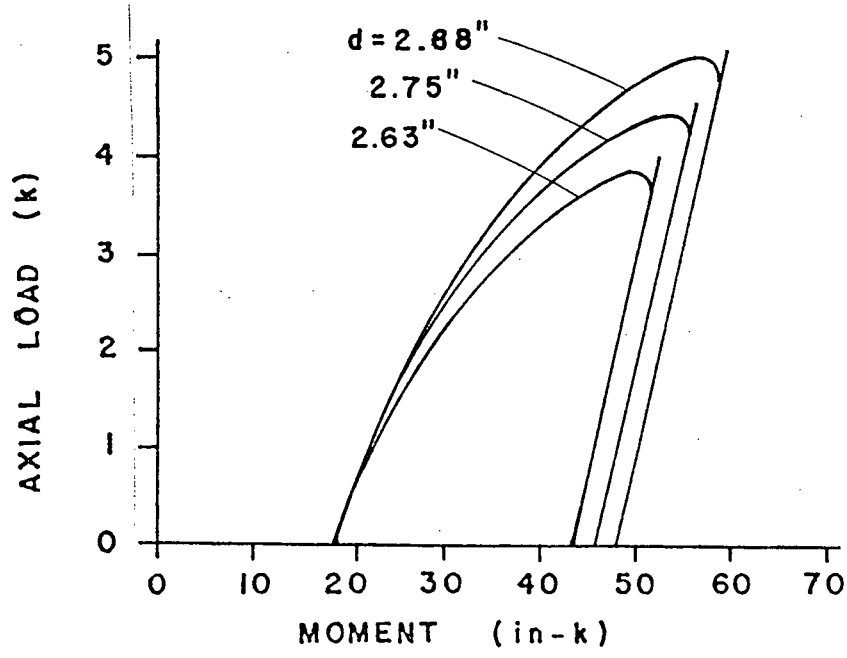


FIGURE 3.6 a

AVERAGE STIFFNESS FOR VARIATIONS IN STEEL LOCATION

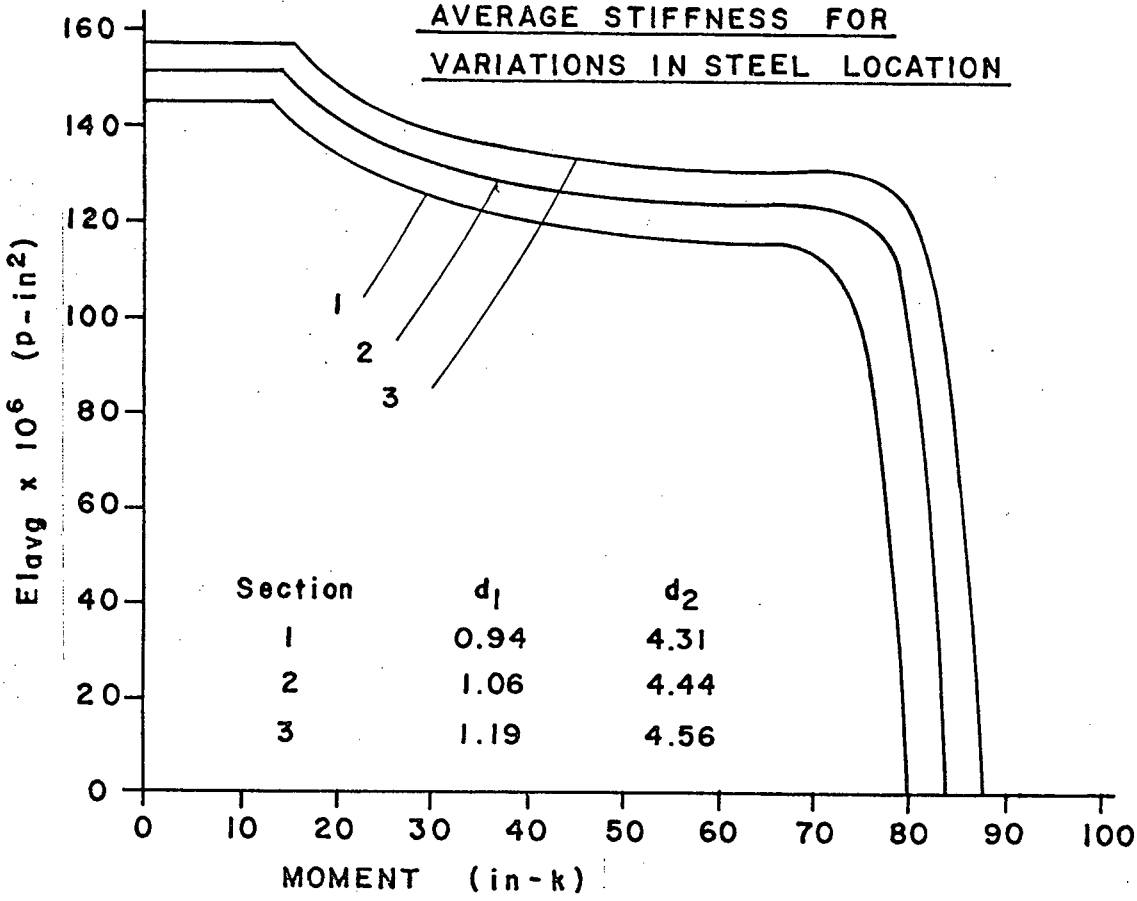


FIGURE 3.6 b

AXIAL LOAD FOR VARIATIONS IN STEEL LOCATION

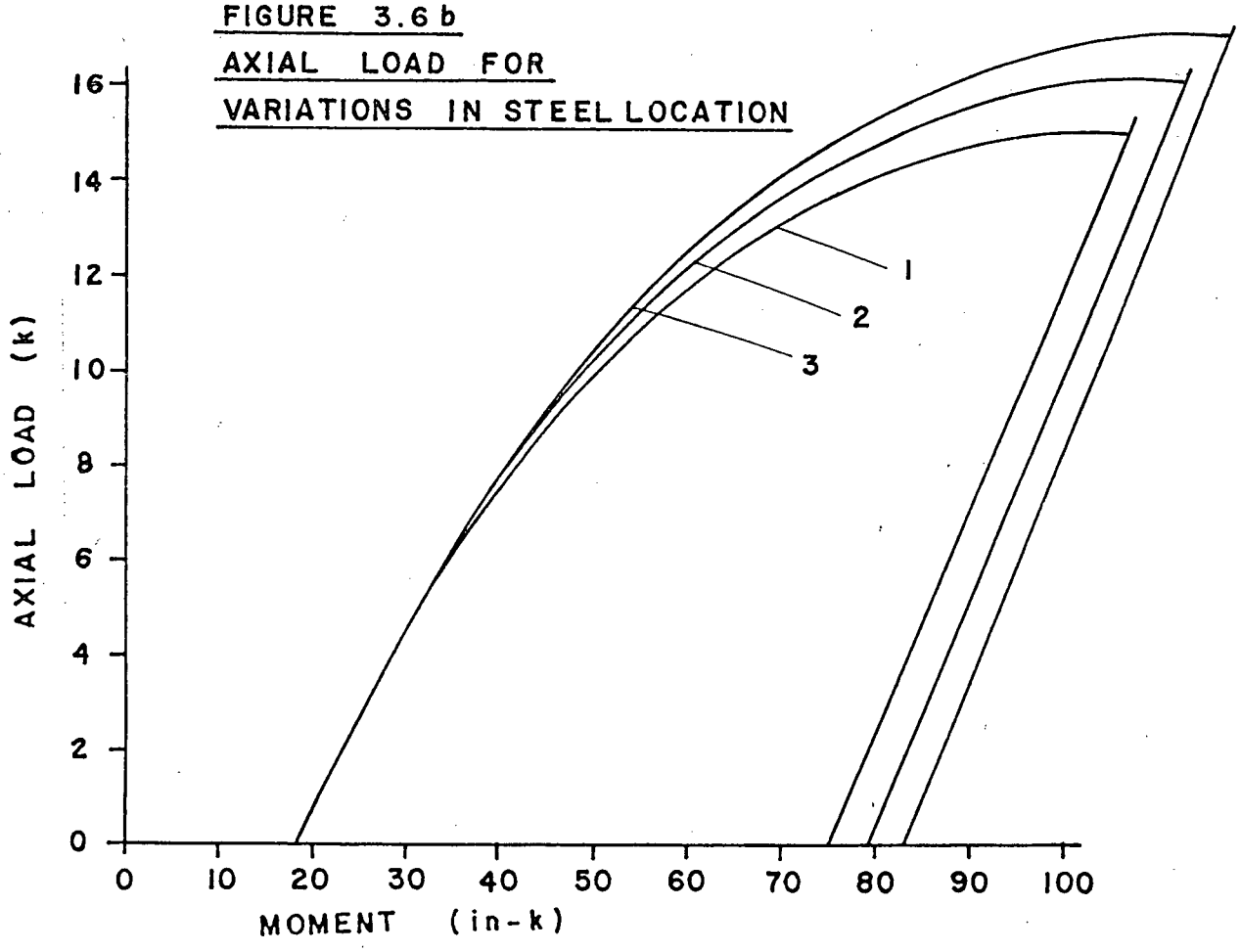


FIGURE 3.7a
EFFECT OF PRESTRESSING
ON AVERAGE STIFFNESS

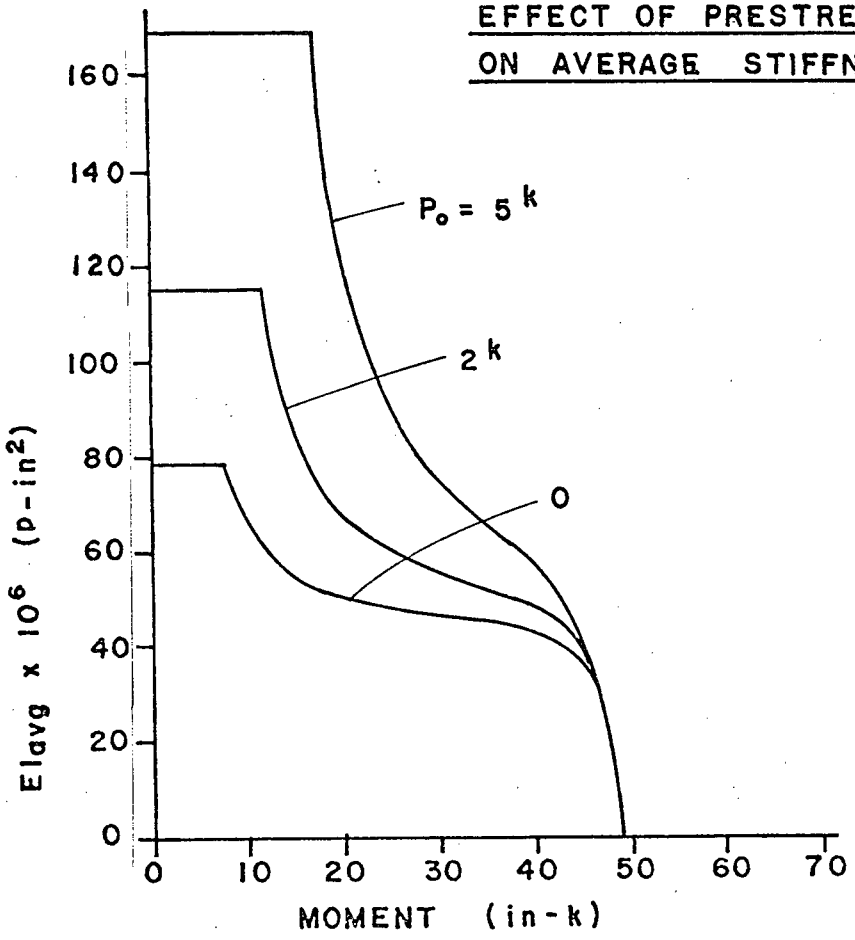
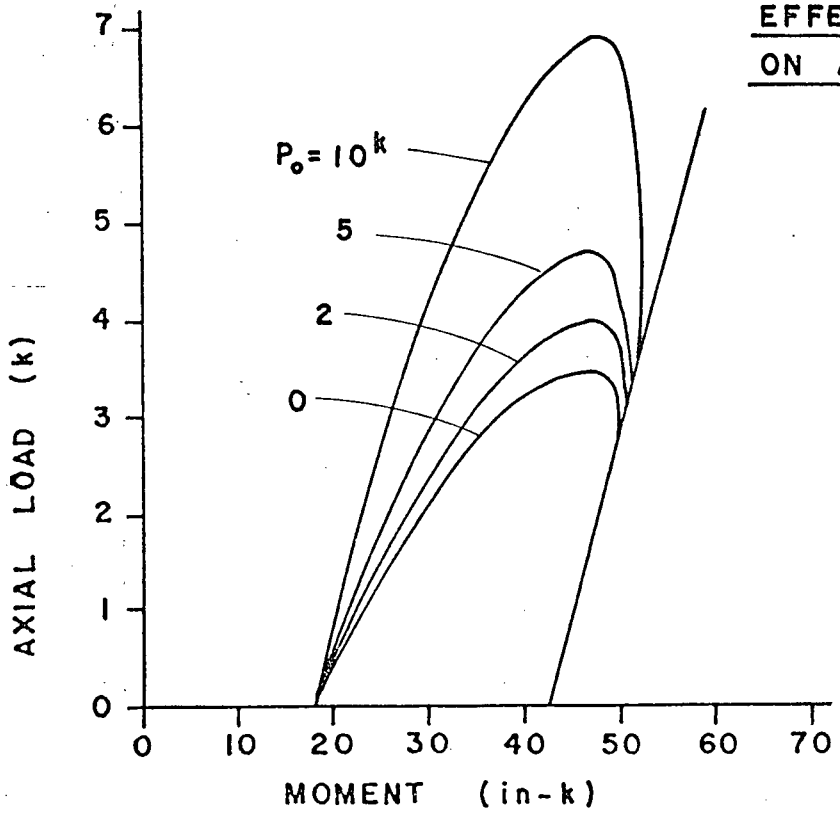


FIGURE 3.7b
EFFECT OF PRESTRESSING
ON AXIAL LOAD CAPACITY



The amount of reduction increases with increasing axial load and reflects the fact that failure due to crushing of concrete is less predictable than yielding of reinforcing steel.

The codes also suggest that the stiffness used to compute deflections and thus, the total moments, should also be reduced by the Ø factor. The effect of Ø on average stiffness is shown in Figure 3.8.

If this reduced stiffness is used (in actual fact curvature is increased by the inverse of Ø), the computed load capacities will naturally be less as indicated in Figure 3.9. Line A is the design load curve for a panel with no reduction in section stiffness, and line B is the same panel with Ø applied to the curvature only. Line C is the final result where Ø is used to reduce the axial load and moment computed for line B.

The effect of this double reduction is substantial as can be seen in Figure 3.9, but is considered necessary particularly in view of the effects of small variations in placement of reinforcing steel (see Section 3.5).

3.7 Section Thickness

Panel thickness should be considered along with placement of reinforcement when assessing the effects on bending stiffness and load capacity. As was illustrated in section 3.5, reinforcing steel placed in two layers can greatly improve the performance over a single layer of steel in the middle of a section of the same total thickness.

FIGURE 3.8
EFFECT OF Ø ON
AVERAGE STIFFNESS

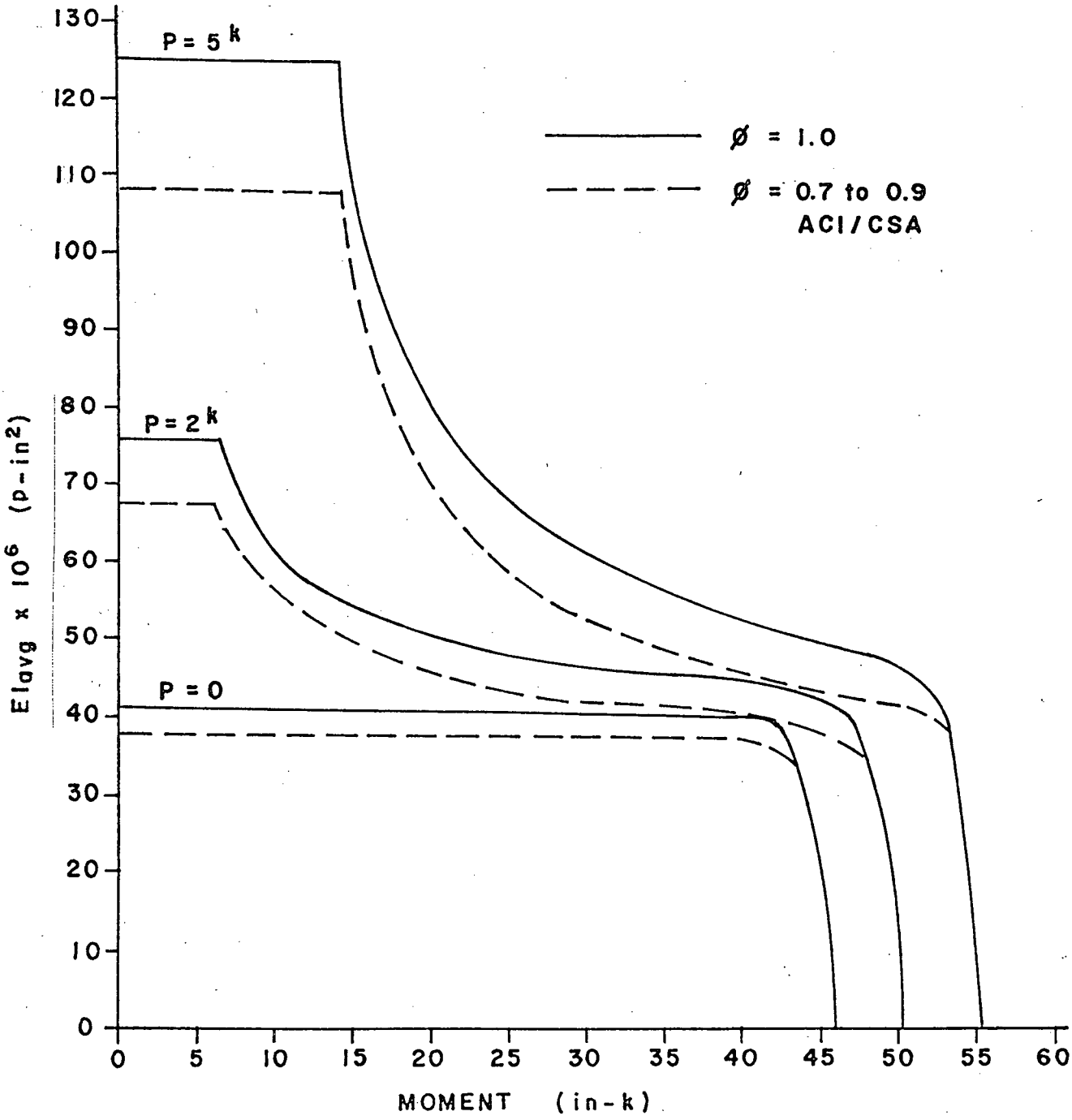
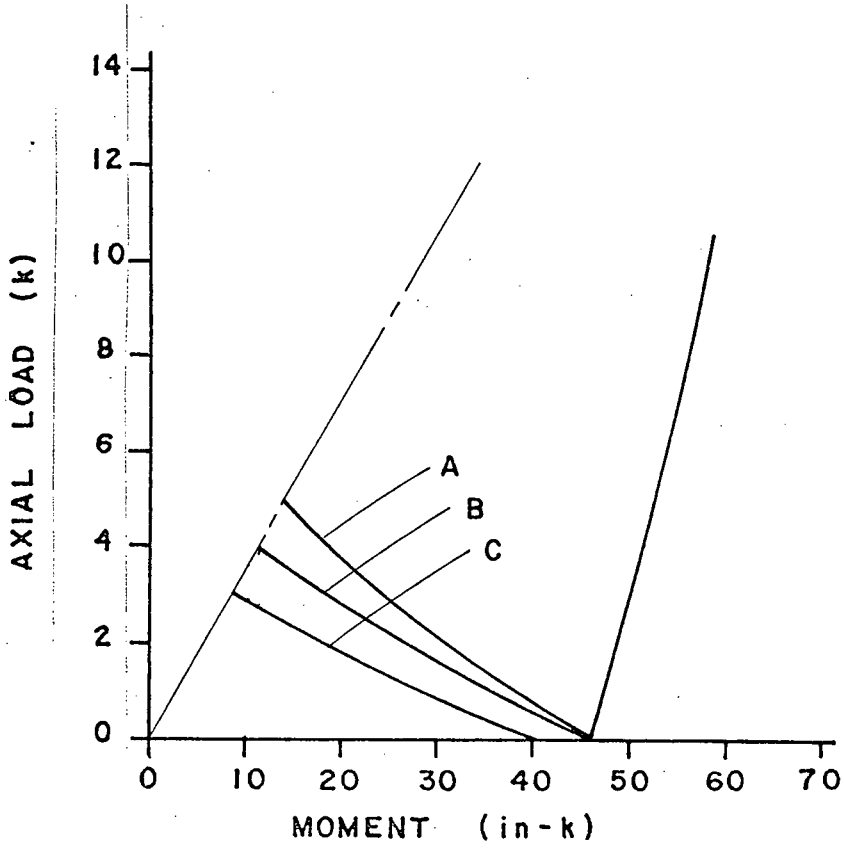


FIGURE 3.9
EFFECT OF Ø ON
LOAD MOMENT CURVES



Since most of the tilt-up panels produced are reinforced with only one central layer, it is worthwhile to investigage the effect of varying the concrete thickness while holding the reinforcement constant. Figure 3.10(a) compares the stiffness of a 5½, 6, 6½, 7 and 7½ inch panel with the same total area of steel in the centre of the cross section.

The trend indicated is that stiffness increases at a rate approximately proportional to the square of the panel thickness or more correctly by the square of the effective depth "d" to the reinforcement. Ultimate moment also increases but only linearly with the depth.

The effect on axial load capacity is shown in Figure 3.10(b). The increase in the peak axial load can be attributed to both the increase in section stiffness as well as to the increase in ultimate moment. It appears to be a function of the depth to the reinforcement raised to a power of at least 3 and possibly as much as 3.5. This, of course, would vary with panel height, end eccentricity and lateral load. It would be conservative, however, to say that increases in axial load capacity are a function of the square of panel thickness ratios for the same overall height.

Conversely, of course, this means a reduction in capacity proportional to square or more of the thickness, if the panel is accidently cast a bit too thin.

FIGURE 3.10 a

AVERAGE STIFFNESS FOR VARIATIONS IN SECTION THICKNESS

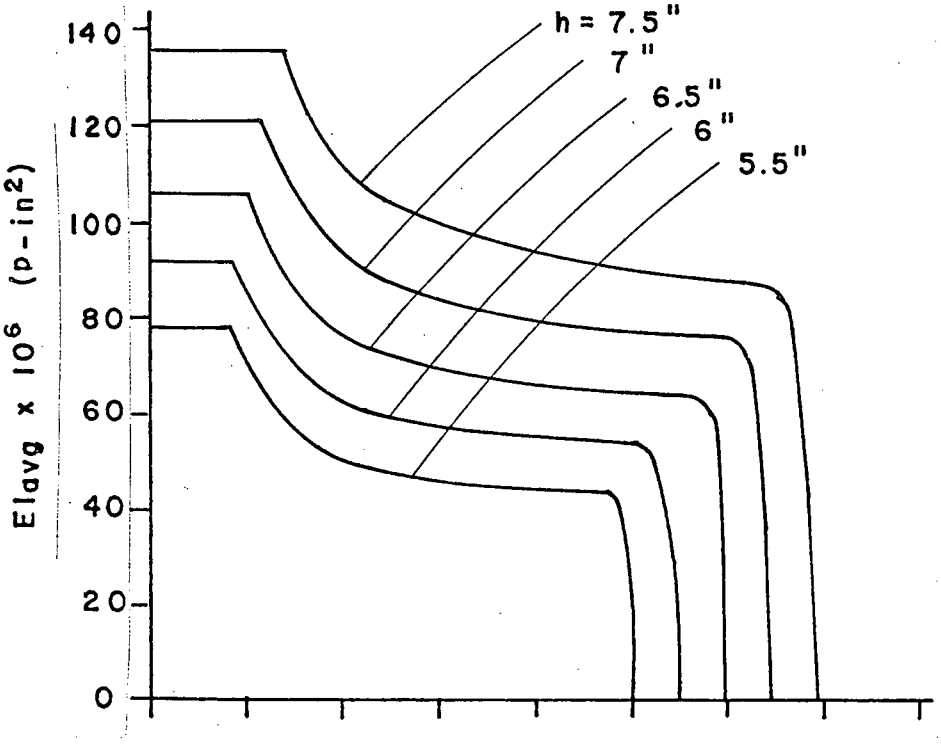
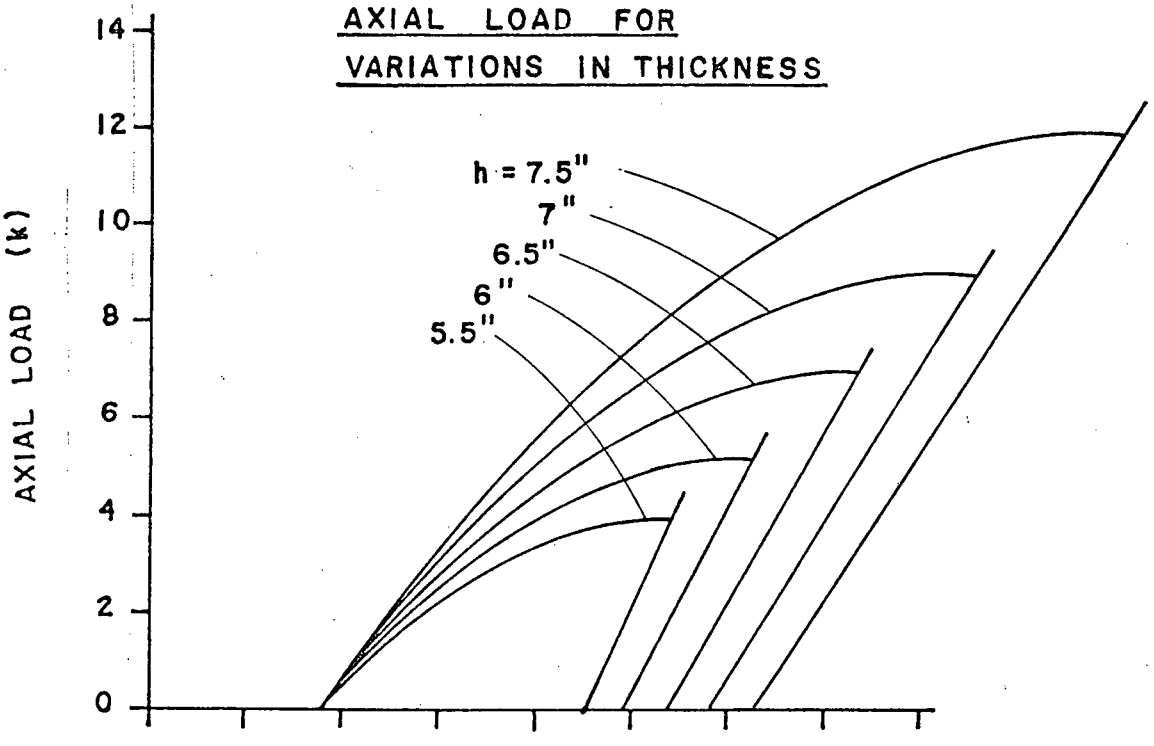


FIGURE 3.10 b

AXIAL LOAD FOR VARIATIONS IN THICKNESS



3.8 Panel Height

The limitations on panel height, or more generally height to thickness ratios, are of interest to all engineers and have often been the topic of lengthy debates by code committees and building officials. There have been attempts to set upper bounds on slenderness ratios based on the codes written for other materials such as structural steel.

Slenderness ratios such as unsupported height/thickness or height/radius of gyration of the gross concrete section are meaningless without consideration of the amount and location of reinforcing steel, the level of axial compression, end eccentricities and lateral load.

The charts in Appendix D indicate that variations in end eccentricity and lateral load can greatly affect the load carrying properties of the panel. Two layers of reinforcing steel instead of one central layer will almost double the capacity for the same overall panel thickness.

3.9 End Fixity

Often a wall panel is continuous over one or more interior supports such as floor and foundations and can be considered as partially restrained at one end. This will usually result in an increased axial load capacity above the simply supported condition.

The computer programs developed were not generally applicable in assessing the effect of end fixity for the following reasons:

a. Horizontal deflections at the top (roof) will now induce secondary moments into the panel. (In simply supported panels the moment is independent of roof deflection.)
b. Lateral restraint provided by the footing is always questionable and may be insufficient to prevent small displacements.
c. Rotational stiffness of other members framing into the joint is unknown.
d. The effect of lateral soil pressure for panels extending below floor slab level is difficult to assess.
e. Self-weight of the panel complicates the analysis further particularly for a. and b.

As a result, the effect of end fixity can at best only be approximated. A popular approach is to assume some reduced or effective length and then to analyze the panel as simply supported.

The effective length of an elastic beam column fixed against rotation at each end would be of the order of 50% of the total distance between supports. Normally, only one end is restrained and this cannot be considered as 100% fixed. It is, therefore, common practice to assume not less than about 85% of the unsupported span although in most cases a full 100% is conservatively adopted for all conditions.

4. DESIGN METHOD

4.1 Description of Design Procedure

The strength of thin concrete wall panels subjected to combined axial and lateral loads can be obtained by a detailed computer analysis which takes into account the variations in section stiffness with axial compression and bending. Since this is not always readily available to the designer, an approximate hand analysis is desired.

The method presented in this manual consists of a series of design charts covering a broad range of panel thicknesses and reinforcing configurations. The maximum moment due to factored applied loading and design eccentricity, but excluding slenderness effects is calculated and compared to the allowable values plotted on the charts. The section properties can then be adjusted until a satisfactory design is reached.

4.2 Load Capacity Charts

A total of 48 design charts has been prepared to assist in the design of tilt-up wall panels (Appendix B). All loads and section properties are in terms of SI (metric) units.

Each chart is applicable to a unique cross section. Three thicknesses were selected: 140 mm, 165 mm and 190 mm with the reinforcing steel placed in one layer in the middle of the section (A1 to A24) or in two layers 20 mm clear of each face (B1 to B24). The area of reinforcing ranges from 300 mm^2 to 1 400 mm^2 in a 1 000 mm wide section.

A 28 day concrete cylinder strength of 25 MPa was used. The yield stress of the reinforcing steel conforms to the new metric standard of 400 MPa.

Each chart contains interaction curves for failing values of applied axial load and unmagnified moment for a number of unsupported heights. The design procedure consists of entering the chart with the axial load applied at the point of maximum moment (usually about mid-height). If the moment given by the chart for the design height if panel is greater than the maximum moment due to all effects including wind and eccentricity and initial out-of-straightness, but excluding slenderness magnifications, then the section can be considered adequate. Note that all loads are ultimate (factored) and the capacity reduction factor Ø has been included in the charts.

For a more detailed description of the development of these charts, see Section 2.

4.3 Loading Conditions

a. Lateral loads: usually wind pressures control the design although seismic accelerations may in some cases be significant. Local codes should be consulted, but in no case should a lateral pressure of less than 0.5 kN/m^2 be used (positive or negative).

b. Axial loads: these are usually uniformly distributed line loads along the top of the panel. For light point loads at small eccentricities, it is sufficient to substitute an equivalent uniformly distributed load by

dividing the load by the spacing. Where axial load is large (50 kN), the width of influence should be limited to the load length plus approximately 12 times the panel thickness.

c. Eccentricities: axial loads will always be applied at some eccentricity to the centreline axis of the panel, either intentionally or due to bearing irregularities. A minimum eccentricity of one-half the panel thickness is recommended for design computation where the effect is additive to the wind load and zero where a reduction of total moment would otherwise occur. Eccentricity at the bottom is assumed to be zero.

d. Self-weight: the effect of panel self-weight on moment magnification can be approximated by assuming that a portion of the total weight acts at the top as a concentric axial load. Since the critical section usually occurs at or slightly above mid-height, it is conservative to use one-half of the total panel weight.

e. Load combinations: there are five load combinations that may be considered, according to the National Building Code of Canada, and each should be investigated separately:

$$U = 1.4D + 1.7W$$
$$U = 1.4D + 1.8E$$
$$U = 1.4D + 1.7L$$
$$U = 0.75\ (1.4D + 1.7L + 1.7W)$$
$$U = 0.75\ (1.4D + 1.7L + 1.8E)$$

Usually the first condition will control the design although the third is significant for short panels with large eccentric axial loads.

For the common case of large wind load moments and small axial loads, the maximum moment will occur very near mid-height of the panel. As the axial load and end moment increase, this point will shift towards the top. The design charts require that the maximum moment, wherever it may occur, be plotted against axial load.

4.4 Material Properties

The selection of material properties should be based largely on local conditions and practice, and is usually governed by cost considerations.

The specified 28 day concrete strength should be in the order of 30 to 35 MPa such that sufficient flexural strength is available for lifting. Many contractors will ask for flexural strength tests just prior to lifting to confirm the strength requirements. The compressive strength of concrete is relatively unimportant as long as a minimum of 25 MPa at 28 days is reached.

The grade of reinforcing steel should always be hard grade, 400 MPa unless it is only required for temperature stresses. Mixing grades of the same size of rebar on one jobsite should be avoided.

The use of lightweight aggregates causes a decrease in load capacity in the order of about 5%, however due to the reduced self-weight these panels will usually carry greater roof and wind loads than if normal weight concrete is used.

4.5 Effective Panel Height

It is recommended that all panels be considered as pinned at each end unless the bottom support is fixed so that all rotation is prevented. In that case, a reduced effective length may be used in the order of about 85% to 90% of the clear distance between actual supports.

Practical limits on the unsupported height of a wall panel must be based on section thickness, reinforcing configuration and loading condition. Figure 4.1 may be used as a guide when making preliminary selections of panel designs. This diagram has been established on the basis of the following conditions:

a. Concrete strength 25 MPa.
b. Reinforcing steel yield 400 MPa.
c. Steel area 800 mm^2/m in the centre or 800 mm^2/m each face (20 mm clear).
d. Wind load Wu = 1.75 kN/m^2 (positive).
e. Axial load Pu = 6.5 kN/m^2 plus ½ self-weight x 1.4.
f. End eccentricity e = o.
g. Initial deflection 20 mm.

4.6 Effect of Creep and Initial Deflections

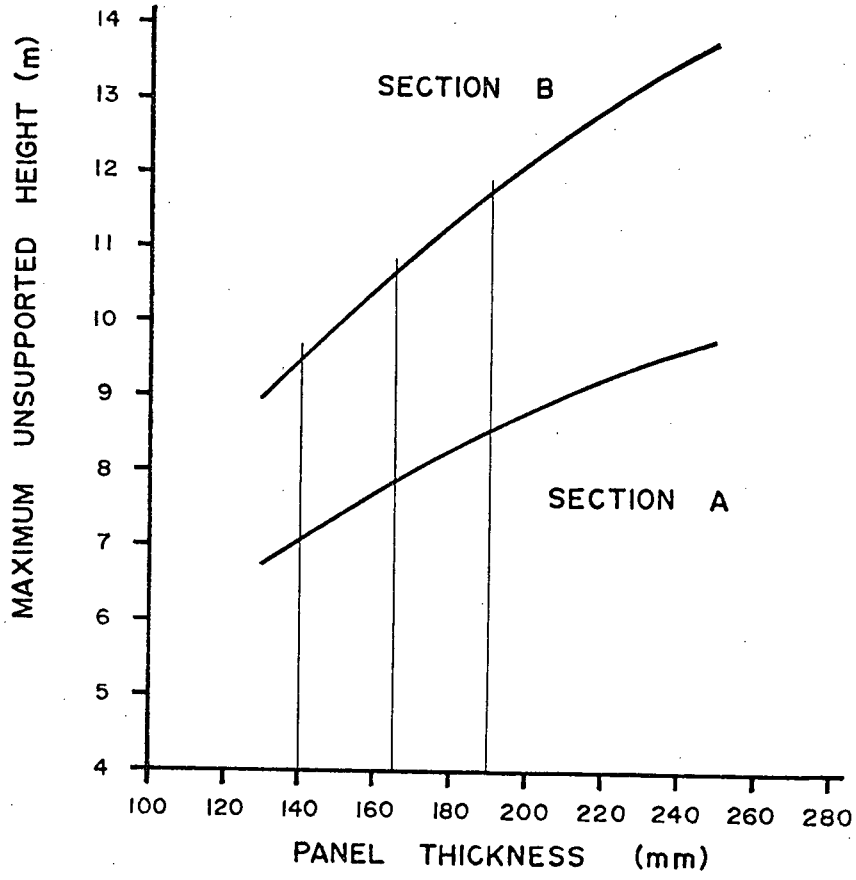
Long term deflections caused by creep are not normally considered since dead load deflections are usually quite small compared to those from wind load.

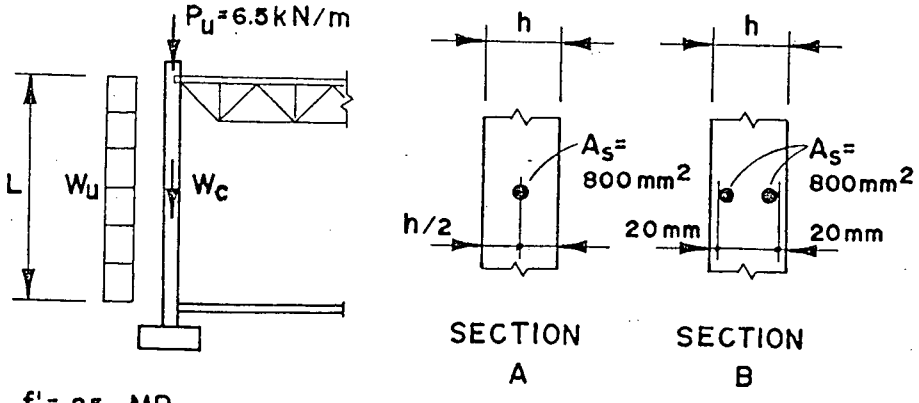
Initial out-of-straightness caused by variations in shrinkage or by casting on uneven floor slabs can be approximated by adding a small deflection at mid-height in the order of 20 mm.

FIGURE 4.1

CONCRETE WALL PANEL

HEIGHT – THICKNESS RELATIONS





f'_c= 25 MPa
f_y= 400 MPa
W_u=1.75 kN/m^2

Deflections at roof level due to the flexibility of the roof diaghragm have no influence on slenderness effects unless there is fixity at either the top or bottom of the panel.

4.7 Capacity Reduction Factors

In the ACI design procedure, the factored moments are increased by the moment magnifier and compared with the ultimate moment reduced by the capacity reduction factor. Because variation in material properties and dimensions influence stiffness and therefore slenderness effects, the capacity reduction factor appears a second time in the moment magnifier. A parallel procedure has been followed in the present work: in computing the effects of slenderness, the moment rotation relationship or stiffness of the section was reduced by the capacity reduction factor. The latter was subsequently applied a second time to reduce both axial load and moment before preparation of the charts.

The actual value of Ø was based on the recommendation by ACI 318 and CSA A23.3

$$\emptyset = 0.9 - 2.0 \frac{Pu}{f'cAg} \geq 0.7$$

4.8 In-Plane Shear

In-plane shear is only significant for high, narrow panels with substantial shear loads applied at the top. This sometimes occurs in very long buildings where end walls are required to resist the lateral shear forces from the roof diaphragm. In these cases, the stability of the compressive edge in buckling perpendicular to the plane of the panel is the main point of interest as actual shear stresses would rarely be excessive.

Figure 4.2 shows the proposed method of analysis by which the shear load is converted into an effective self-weight at the outer compressive edge. In most cases, this would involve applying half of the total effective self-weight to the top of the panel. If this edge is stable under the combination of effective self-weight plus lateral wind load, then no further bracing is necessary. If not, a thickened edge beam may be added. The simplest method, however, involves connecting the panels together so that the compression edge of one panel is tied to the tension edge of another and the complete wall acts as a unit.

When a panel subjected to large in-plane shears also has large openings, the effect of in-plane frame action must be considered. In all cases, overturning and sliding should be checked and ties provided to the footing or floor slab as required.

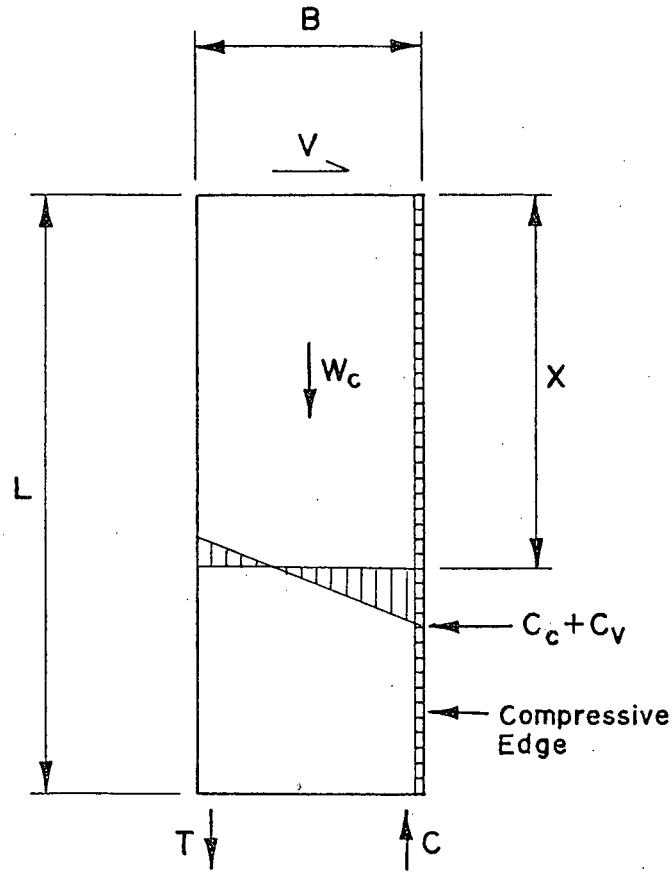
4.9 Panels with Openings

Panels with openings can be analyzed by two dimensional finite element methods, but this is usually expensive and time consuming, and is rarely justified. An approximate one dimensional analysis will give results that are sufficiently accurate for most designs and can be adapted to hand calculations.

Where openings occur, the lateral and axial loads (including self-weight) on the entire panel must be carried by the continuous vertical legs each side of the opening (see Figure 4.3). Often it is only necessary to increase the loads acting on the legs by the ratio of the total panel width to the leg width, although some designers may alternatively wish to use point loads exerted by window and door frames.

FIGURE 4.2

IN-PLANE SHEAR



Compression due to self weight (per unit length)

$C_c = W_c \cdot X$

Max. comp. due to in-plane shear V (per unit length)

$C_v = V \cdot X \cdot 6 / B^2$

Total comp. $= C_c + C_v$

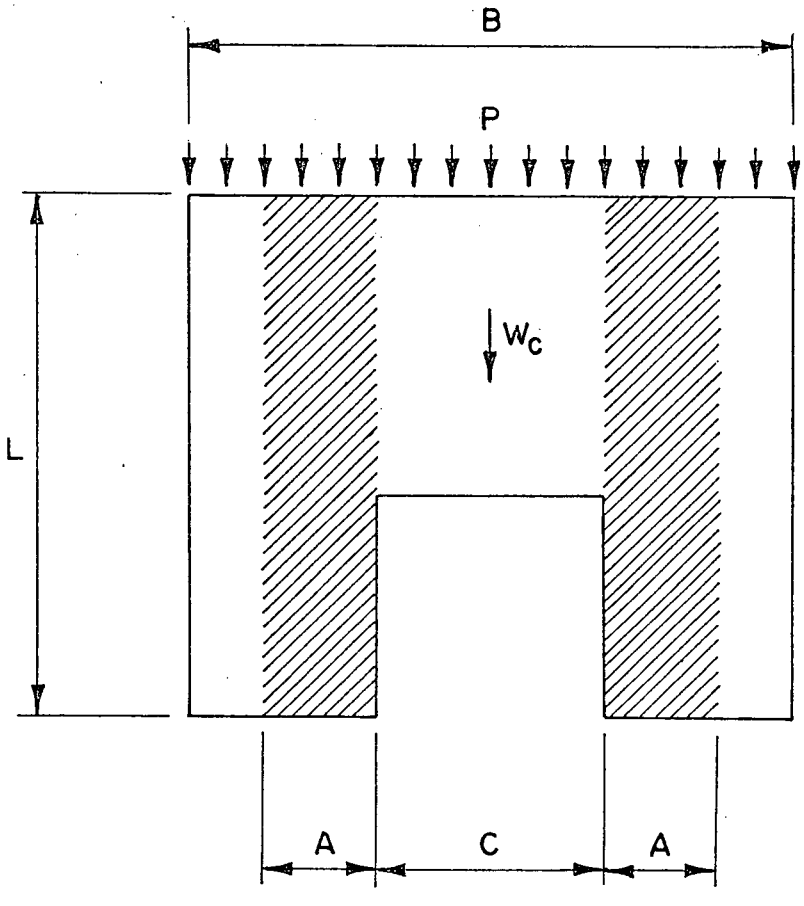
$= (W_c + 6V/B^2)X$

Equiv. self weight (per unit area)

$W_e = W_c + 6V/B^2$

FIGURE 4.3

PANELS WITH OPENINGS



Effective leg width

Max. A = 12 h

Load on each leg

Ratio R = (C+2A)/2A

Axial = PR

Lateral = WR

Self weight = W_c.R

The width of the leg assumed to be carrying these additional loads should be restricted to about 12 times the panel thickness in order to avoid the possibility of local buckling at the edge of the opening.

For very wide openings in excess of 5 m, it is often necessary to provide a thickened pilaster at each vertical edge and a horizontal header beam at the top (and bottom) of the opening.

4.10 Isolated Footings

Isolated or intermittent footings are normally employed in situations where piling is specified. Otherwise, it has been found that continuous strip footings provide a more desirable means of panel support.

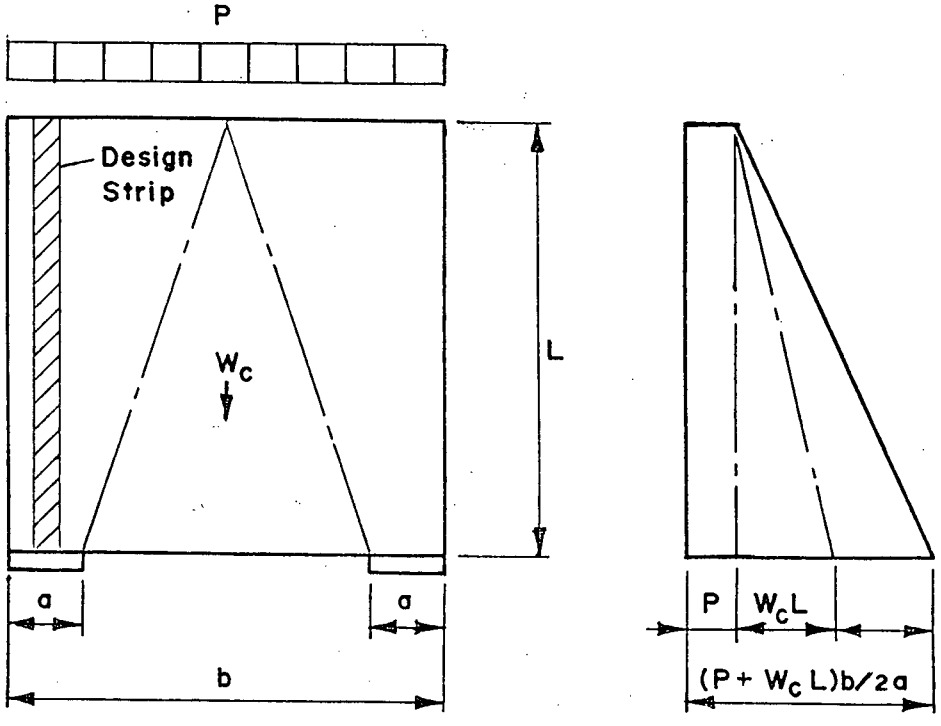
An isolated footing at each end of the panel causes a buildup of high compressive stress near the point of support. The situation can be satisfactorily simulated in much the same way as in-place shear by adding an effective self-weight and checking the panel immediately above the footing. It is common to assume that the load spreads out at an angle of about 30° to the vertical.

Alternatively, the axial compression can be assumed to increase linearly from the top to bottom of the panel as shown in Figure 4.4. The effective panel self weight would be:

$$W_e = W_c + \frac{P + W_c L}{L} (b/2a - 1) \text{ (weight per unit area)}$$

One-half of the total effective self weight would then be applied at the top of the panel.

FIGURE 4.4 ISOLATED FOOTINGS



PANEL ELEVATION

LOAD DIAGRAM ON DESIGN STRIP

5. COMPARISON OF RESULTS

5.1 Experimental Verification of Computer Program

The program developed for this publication was based on the same principles as one used previously for prestressed concrete wall panels. That program was experimentally confirmed by testing (Ref. 4) and by comparison with a limited amount of data from tests by others.

5.2 PCA Design Aid

The PCA design aid for tilt-up wall panels (Ref. 5) contains charts for maximum axial load on wall panels with fixed end eccentricities and lateral loads. A direct comparison is made for the following selected example:

Tables B1 and B2

$h = 5\frac{1}{2}"$ $b = 12"$ $d = 2.75"$

$A_s = 0.0025 \times 12 \times 5\frac{1}{2} = 0.165 \text{ in}^2$

ecc = 2.75"

for w = 30 psf

		PCA		Computed
kL_u/h	L_u (ft)	Coeff.	P_{max}(K)	P_{max}(K)
20	9.17	0.096	25.3	27.5
30	13.75	0.028	7.4	7.5
40	18.33	0.007	1.8	1.9
50	22.92	unstable		unstable

The material properties were identical in both cases, including the concrete stress strain curve. The results obtained are in close agreement, with the differences being attributable to the degree of refinement in computing section properties or in size of load increments on the wall panel model.

The PCA result does not contain an allowance for reduced stiffness by means of the capacity reduction factor and is therefore expected to yield overall higher load capacities. (This reduction was omitted in the "computed" value for the purpose of the above comparison.)

5.3 ACI/CSA Recommendations

A review of the ACI/CSA recommendations (Ref. 2, Section 10; Ref. 3, Section 8) for member stiffness as applied to the design of load bearing wall panels indicates that they cannot be used for most tilt-up design.

The code equations are derived for high axial loads and small eccentricities and are restricted to kLu/r values less than 100 (kLu/h less than about 30). A good number of tilt-up wall panels are now being built where kLu/h is 50 or more, and axial load stresses are generally small in relation to lateral load bending stresses. The ACI/CSA codes require a detailed evaluation of slenderness for these conditions but offer very little to guide the designer.

The design approach described in this paper meets the requirements of the codes with respect to the detailed analysis, and in addition allows the designer to assess the effects of all the factors that may contribute to failure.

6. PROPOSED CHANGES TO ACI/CSA CODES

As was indicated in Section 5.3, the present ACI/CSA codes do not contain adequate provision for the design of tilt-up wall panels. In most cases the limiting slenderness ratios are well below that commonly employed, requiring that a detailed analysis be performed to verify the stability condition. The design charts presented in Appendix B meet the requirements for a detailed analysis and may be used for design, subject to the limitations given in Section 4.

In lieu of the design chart method, a strength calculation can be made where the magnified moment is obtained for a given loading condition. The difficulty lies in obtaining a realistic estimate of section stiffness EI. This should normally be based on a cracked section with full consideration of the effect of axial load. Unfortunately, the calculations are too involved for most design offices unless a computer or programmable calculator is used.

Owing to the fact that the axial load is relatively light, it is usually sufficient (and conservative) to compute the stiffness for the case of zero axial load only and then apply it to all load conditions. It can be shown that the stiffness of a rectangular concrete section with a single layer of reinforcing steel (in the middle) is as follows:

$$EI = E_c b C_d^2 (d - C_d/3)/2 \qquad 6.1$$

E_c = $33W_c^{1.5} \sqrt{f'_c}$

b = width of section

d = depth to reinforcing steel

C_d = depth to neutral axis

$$= \frac{-a + \sqrt{a^2 + 4da}}{2} \quad ; \quad a = \frac{2nAs}{b}$$

A_s = area of steel

n = modular ratio = Es/Ec

For a symmetrical section with two layers of reinforcing steel:

$$EI = E_c\ b\ C_d^2\ (d_2 - C_d/3)/2 + E_s A_s\ (d_2 - d_1)(C_d - d_1) \qquad 6.2$$

The second term is usually less than 5% of the first and the equation can be reduced to:

$$EI = E_c\ b\ C_d^2\ (d - C_d/3)/2 \qquad 6.3$$

d = depth of tension reinforcement

$$C_d = \frac{-2a + \sqrt{(2a)^2 + 4ah}}{2}$$

a = as before

h = total depth of section

These formula are based on the cracked section in which concrete and steel stresses are approximately linear with strain and reinforcing steel has not yet yielded. For this type of section, the concrete strain is of the order of 0.001 when grade 60 reinforcing yields (strain = .0021).

A comparison of the predicted stiffness by equations 6.1 and 6.3 can be made with the charts of Appendix C.

For example, a 5½" panel reinforced in the middle with 0.30 in^2/ft. computed EI = 40.8 x 10^6 P - in^2 (see chart C3 for comparison).

A 5½" panel with two layers of reinforcement 0.30 $in^2/ft.$ each face gives a computed EI = 117.3×10^6 $P\text{-}in^2$ (see chart C9).

Thus, the cracked section stiffnesses as calculated by equations 6.1 and 6.3 can be used in the above formulas for moment magnification giving reasonably accurate but conservative results. The magnification factor obtained with these stiffness values should not exceed 1.5 since deflections become large and instability is eminent.

At higher levels of axial compression, the capacities predicted by this method would result in increasingly uneconomical designs and design charts such as those in Appendix B should be consulted.

7. LIST OF REFERENCES

1. Galambos, T.V., "Column Deflection Curves", Lecture No. 9, Plastic Design of Multi-Storey Frames, Fritz Engineering Laboratory Report No. 273-20, Lehigh University, 1965.

2. ACI Committee 318, "ACI Standard Building Code Requirements for Reinforced Concrete (ACI 318-77)", American Concrete Institute, Detroit, Michigan, 1977.

3. CSA CAN3-A23.3-M77, "Code for the Design of Concrete Structures for Buildings", Canadian Standards Association, 1977.

4. Alcock, W.J., and Nathan, N.D., "Moment Magnification Tests of Prestressed Concrete Columns", PCI Journal, July-August, 1977.

5. Kripanarayanan, K.M., "Tilt-up Load-Bearing Walls", The Portland Cement Association Publication EBO 74.01D, 1974.

Other Relevant Publications

1. MacGregor, J.G., Oelhafen, U.H. and Hage, S.E., "A Re-examination of the EI value for Slender Columns", Publication SP-50-1, The American Concrete Institute.

2. Nathan, N.D., "Slenderness of Prestressed Concrete Beam-Columns", PCI Journal, V. 17, No. 6, November-December, 1972.

APPENDIX A

Notation

A_g	gross area of concrete section
A_s	area of reinforcing steel
B,b	width of panel
C	compressive force
C_d	depth of neutral axis
d	depth to reinforcing steel
e	eccentricity
e_c	concrete strain
e_s	steel strain
E_c	concrete elastic modulus
E_s	steel elastic modulus
f_c	stress in concrete
f_s	stress in steel
h	total depth of concrete section
L	height of wall panel
M	bending moment
n	modular ratio
P	axial force on panel
W	lateral load on panel
W_c	self weight of panel
W_e	effective panel self weight
y	lateral deflection
y'	slope of curve
y"	curvature
Ø	capacity reduction factor

APPENDIX B

LOAD CAPACITY CHARTS

AXIAL LOAD/UNMAGNIFIED MOMENT

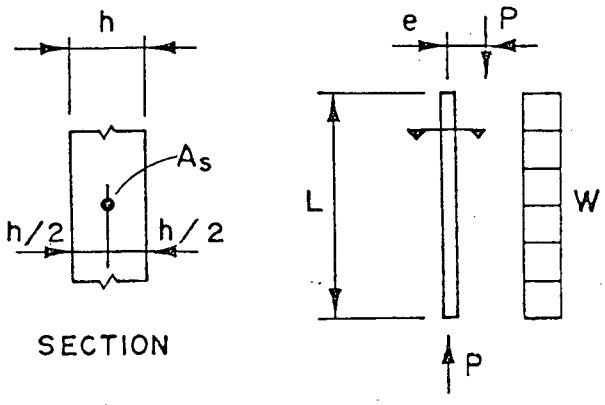
CONCRETE WALL PANEL DESIGN CHART

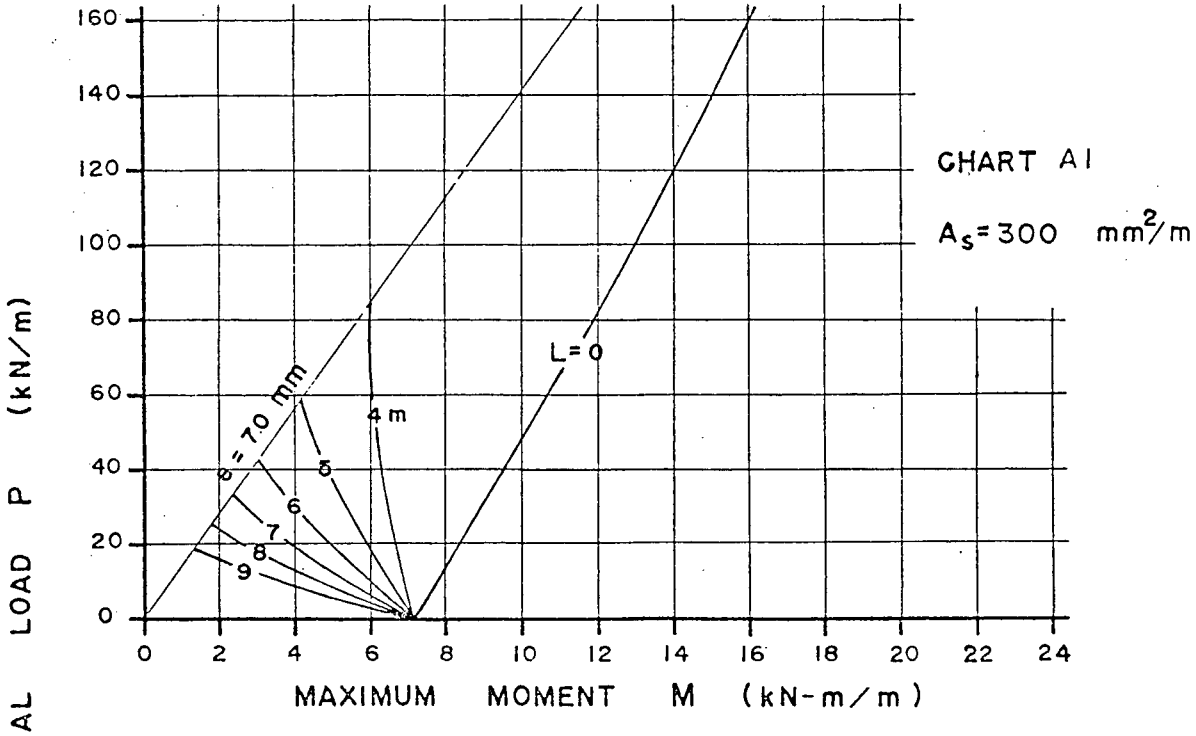
MATERIAL PROPERTIES

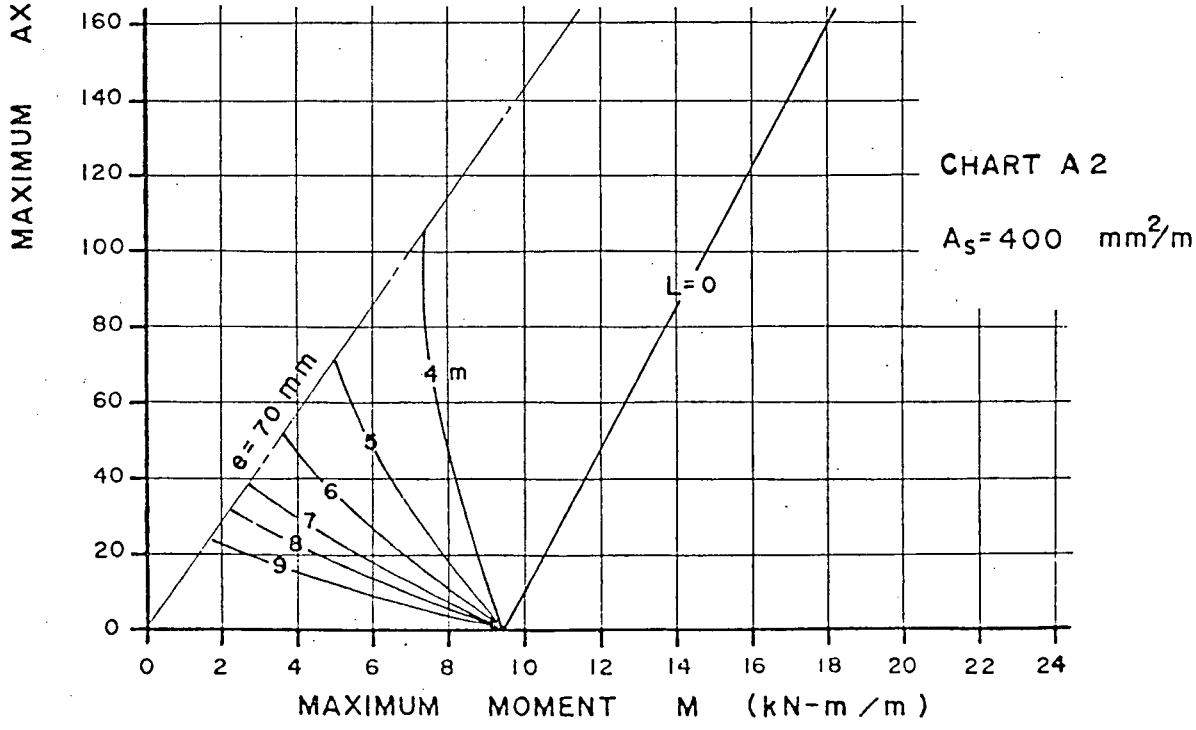
$f'_c = 25$ MPa

$f_y = 400$ MPa

$h = 140$ mm







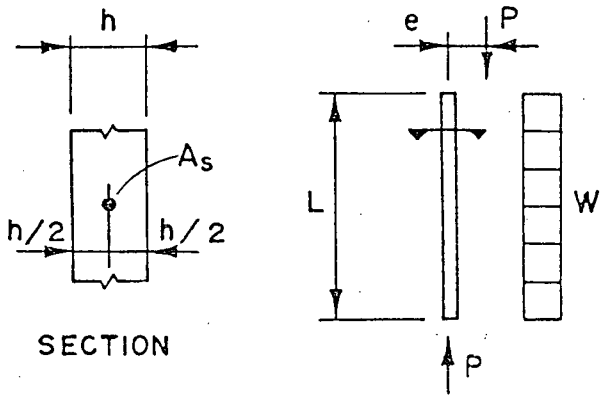
CONCRETE WALL PANEL DESIGN CHART

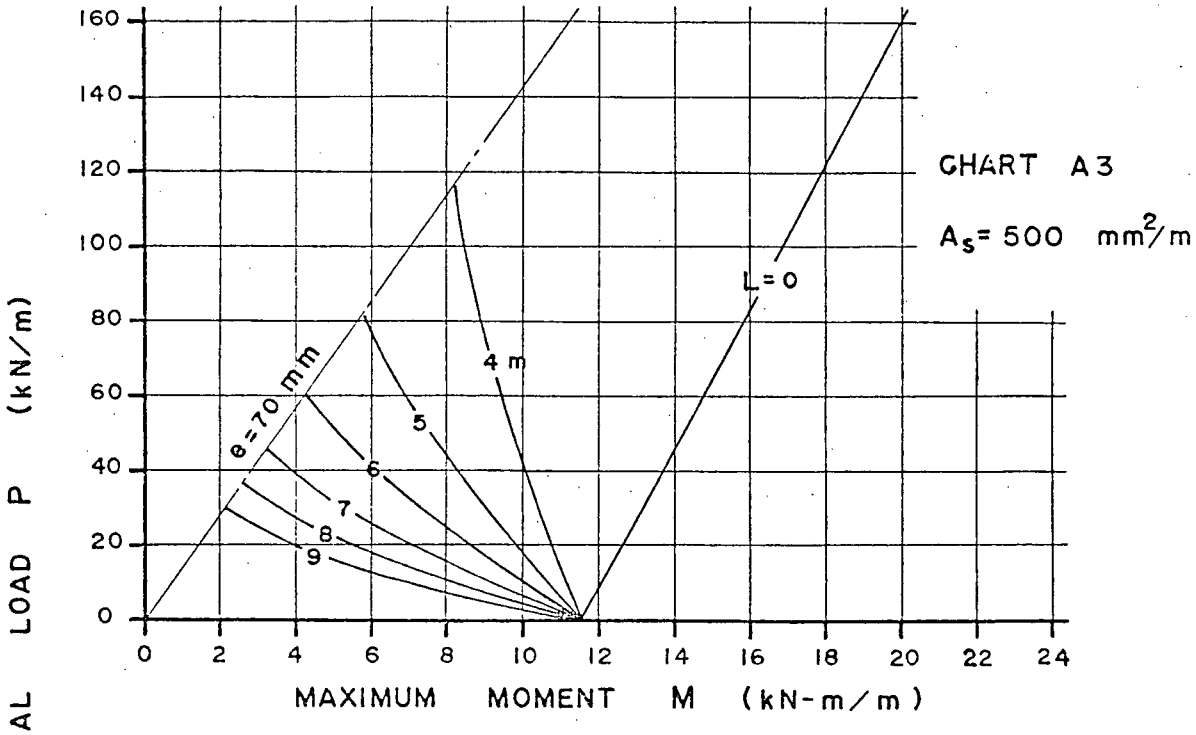
MATERIAL PROPERTIES

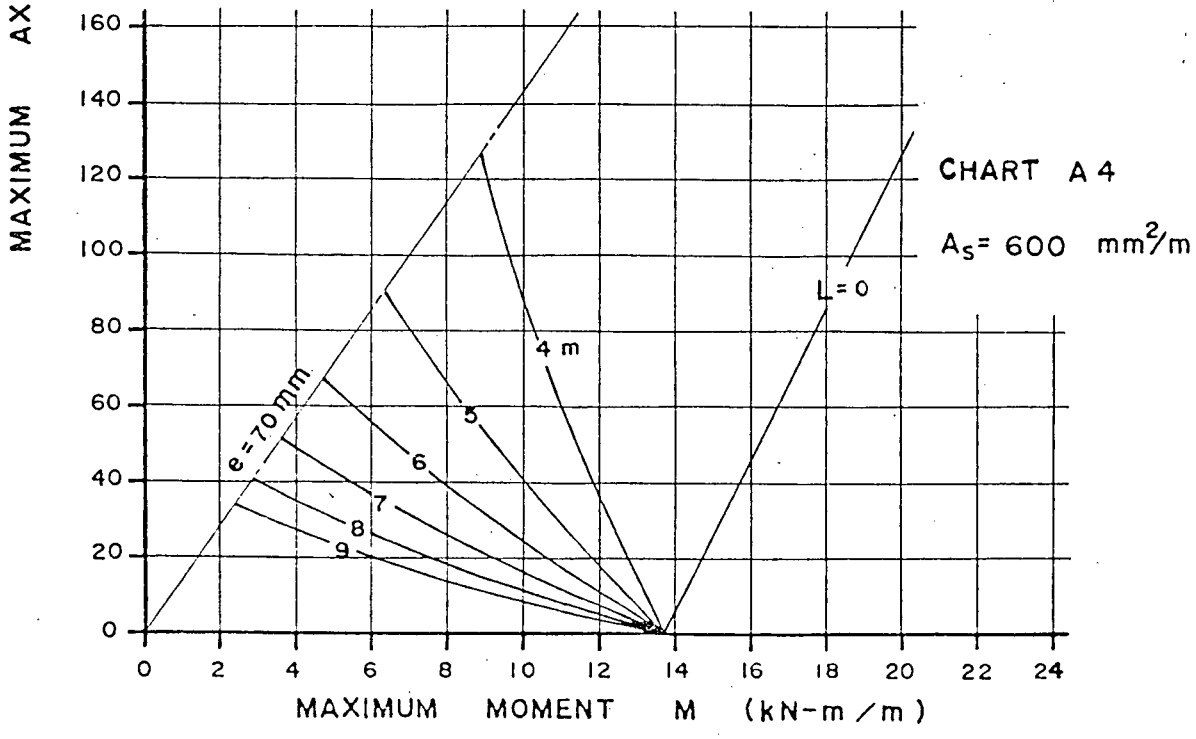
$f'_c = 25$ MPa

$f_y = 400$ MPa

$h = 140$ mm







CONCRETE WALL PANEL DESIGN CHART

MATERIAL PROPERTIES

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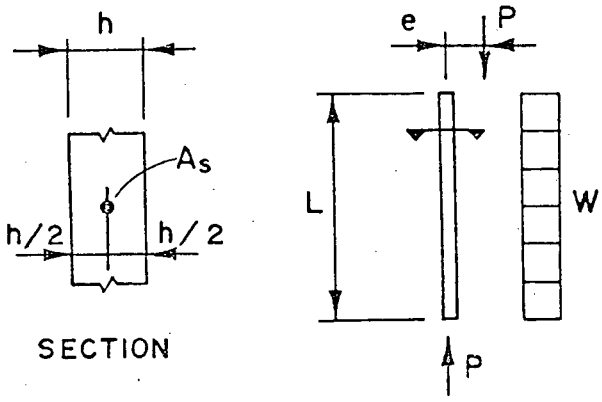
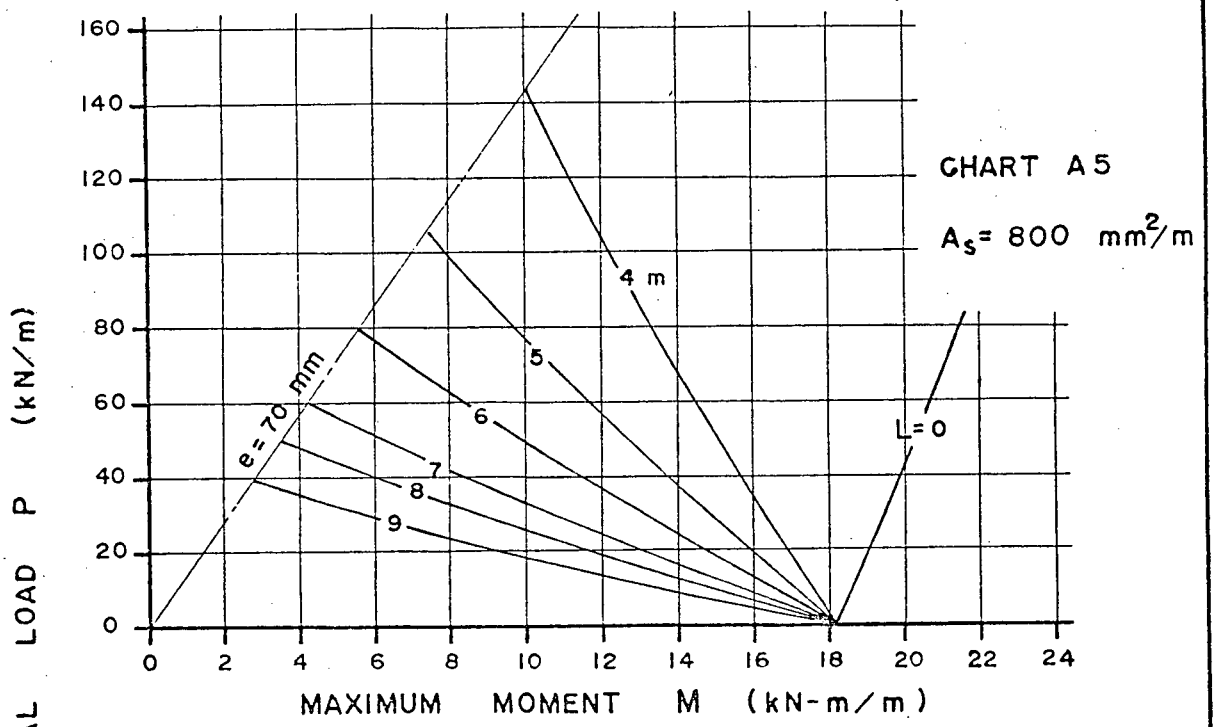
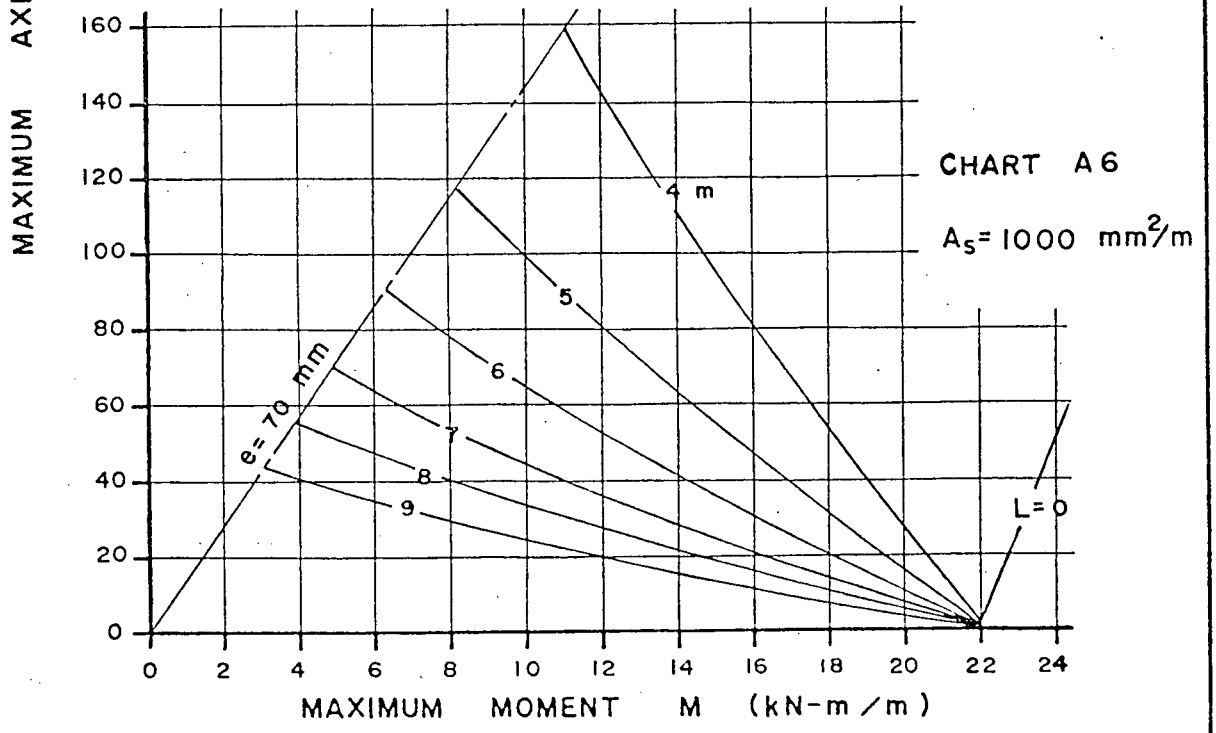
$f_y = 400$ MPa

$h = 140$ mm







CONCRETE WALL PANEL DESIGN CHART

MATERIAL PROPERTIES

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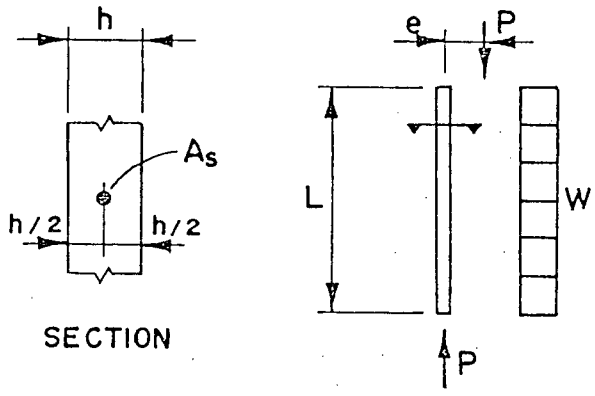
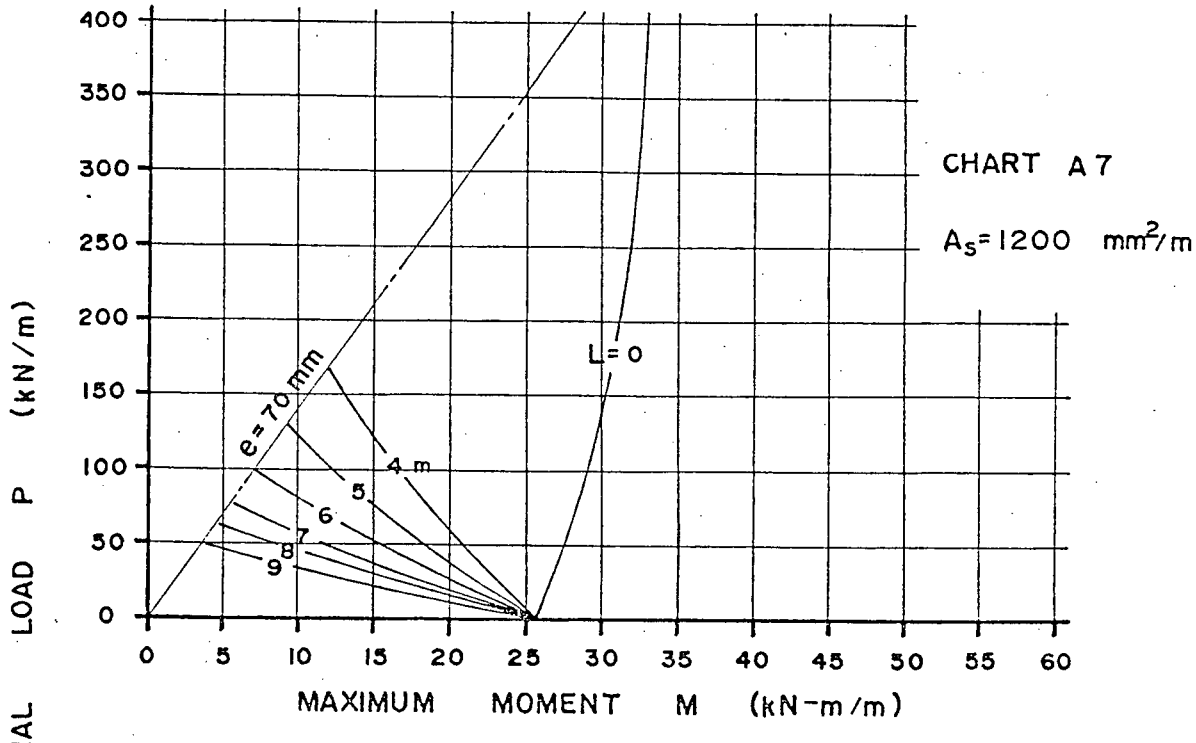
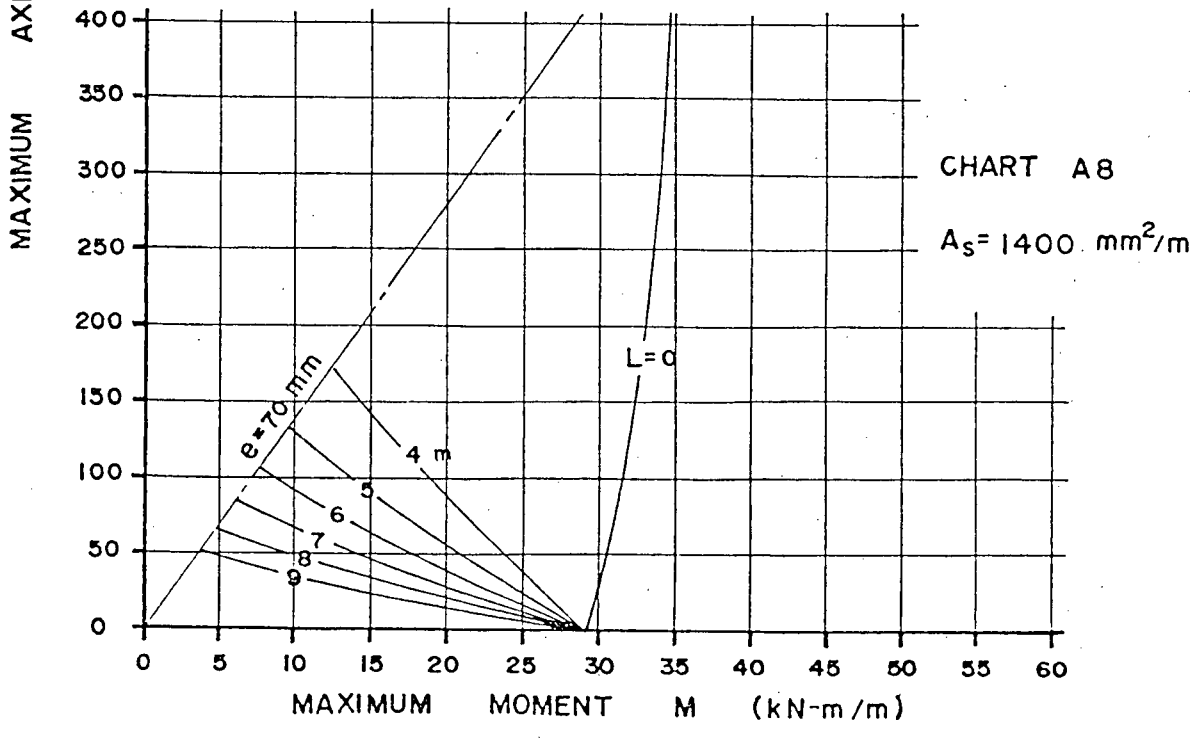
$f_y = 400$ MPa

$h = 140$ mm







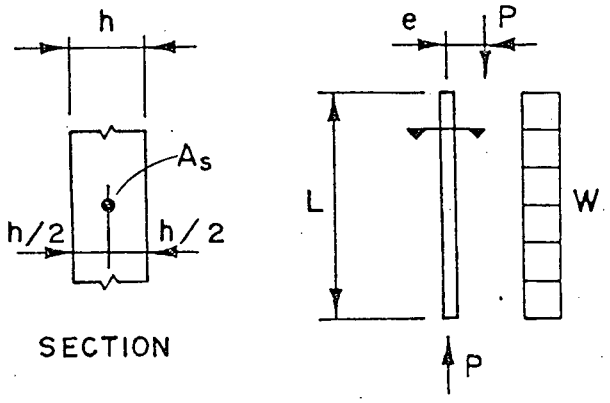
CONCRETE WALL PANEL DESIGN CHART

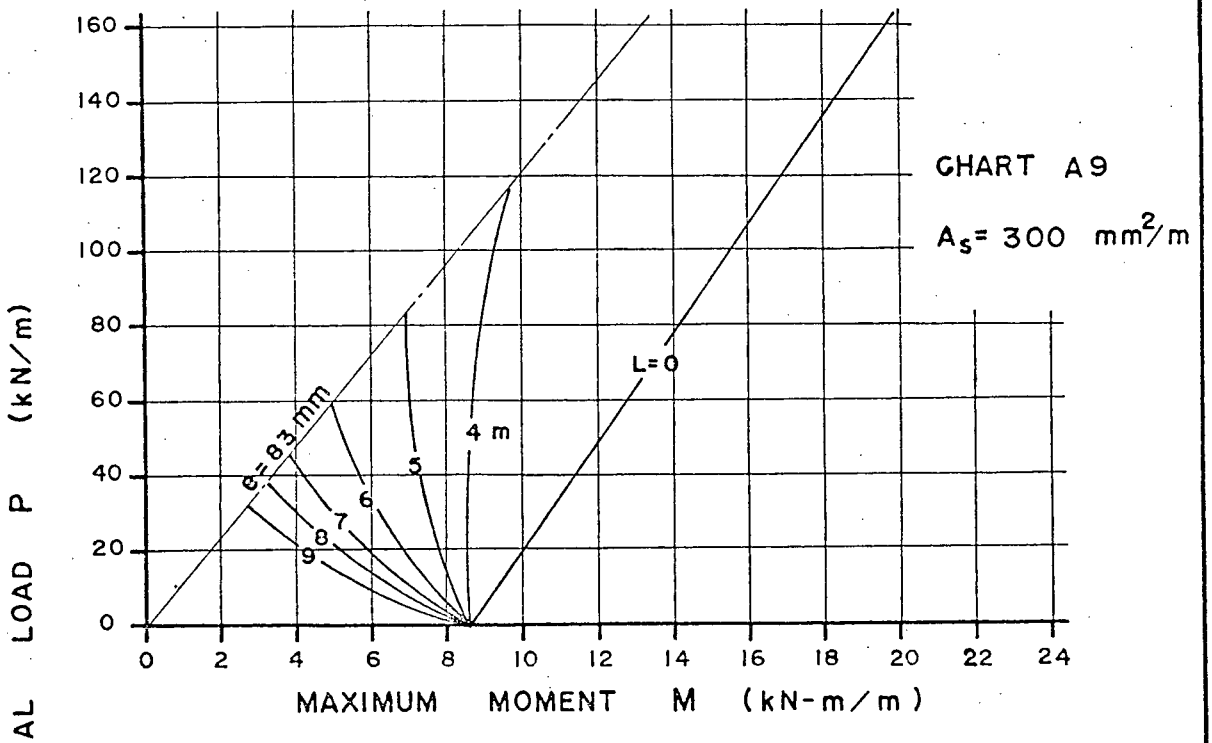
MATERIAL PROPERTIES

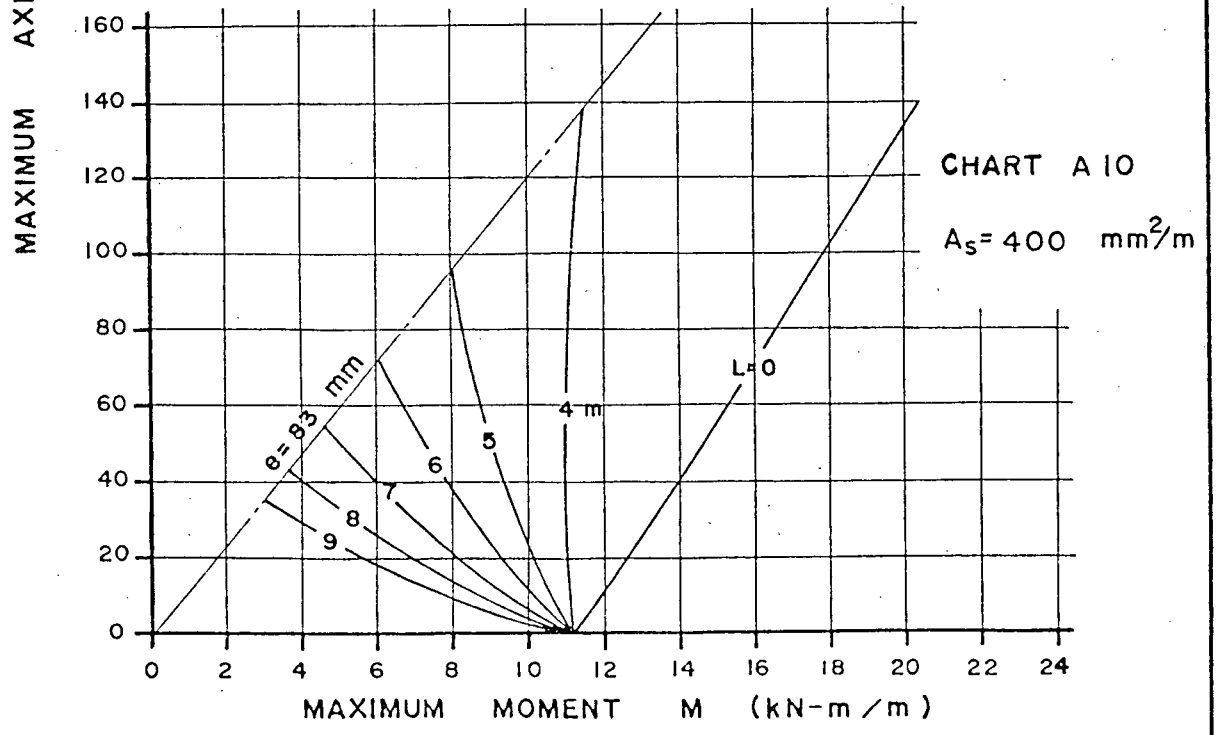
$f'_c = 25$ MPa

$f_y = 400$ MPa

$h = 165$ mm







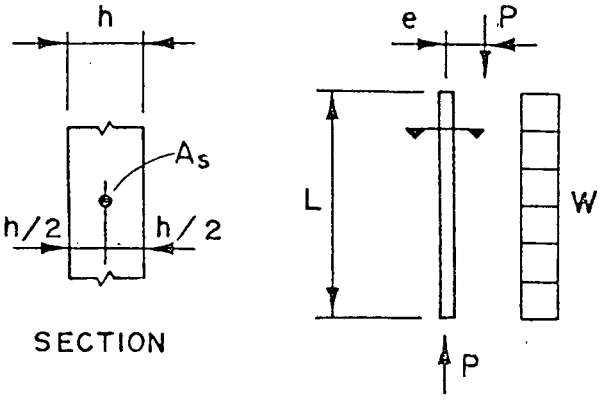
CONCRETE WALL PANEL DESIGN CHART

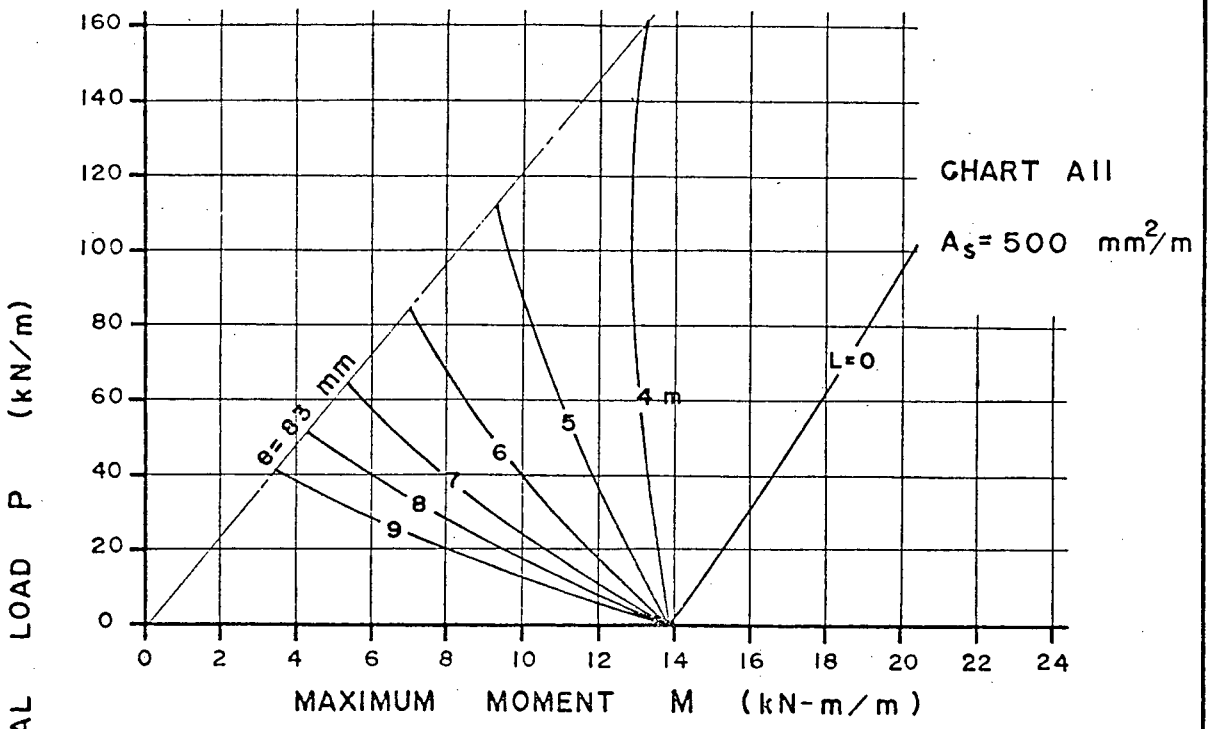
MATERIAL PROPERTIES

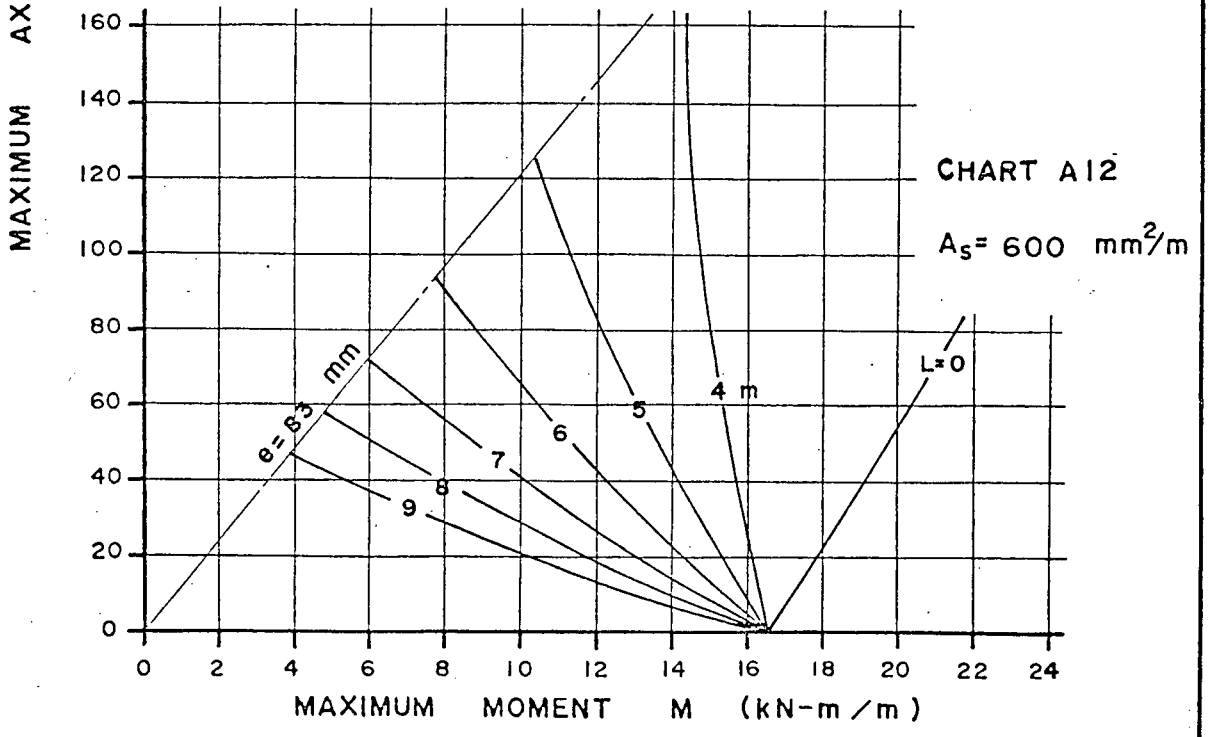
$f'_c = 25$ MPa

$f_y = 400$ MPa

$h = 165$ mm







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MATERIAL PROPERTIES

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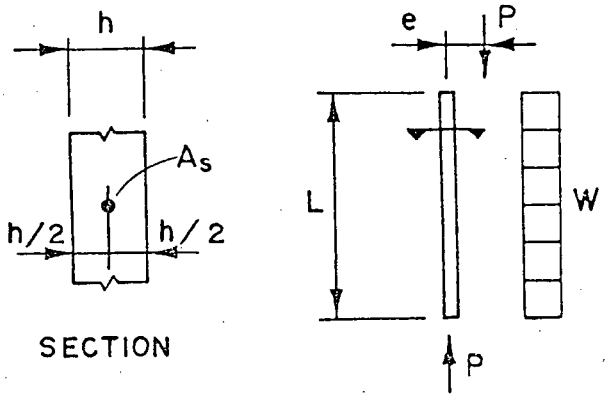
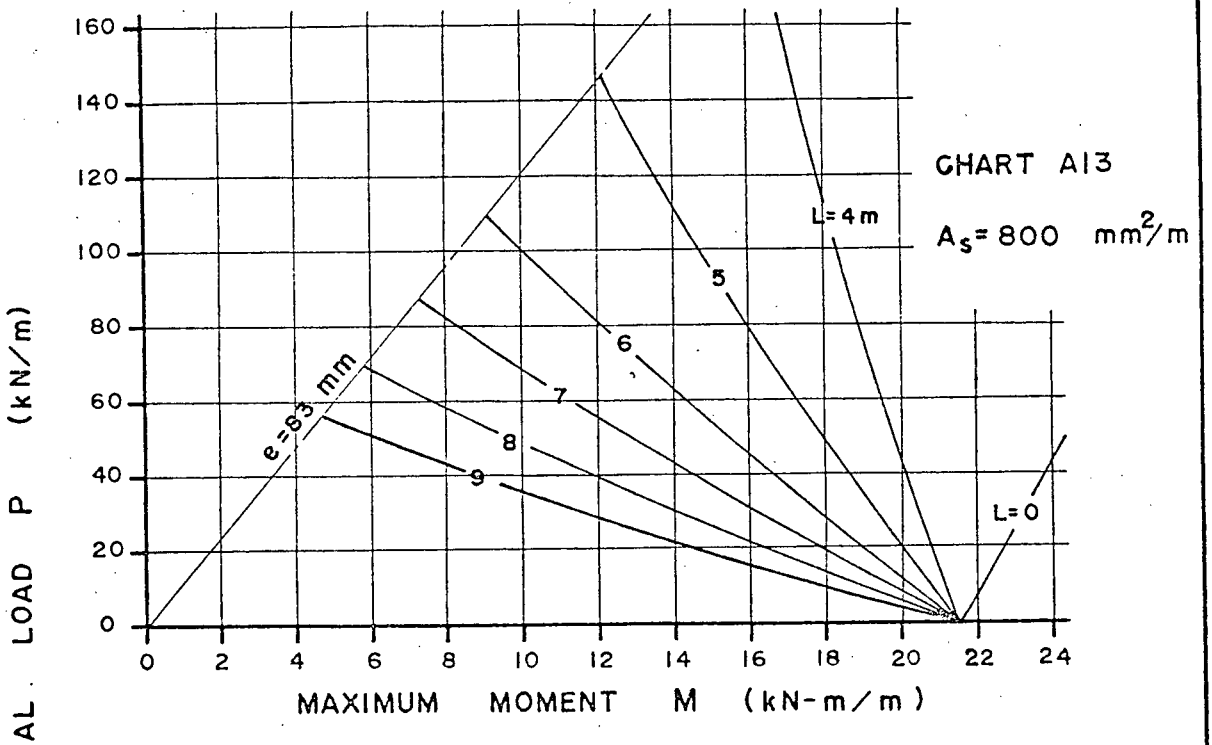
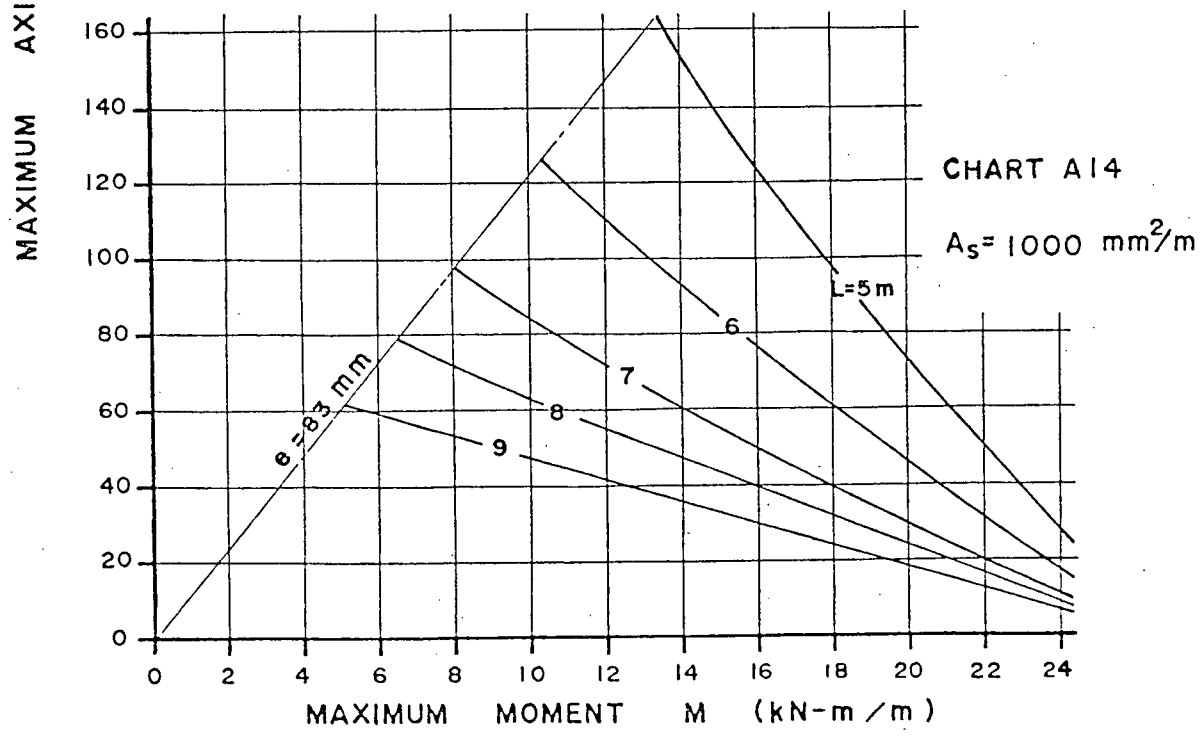
$f_y = 400$ MPa

$h = 165$ mm







CONCRETE WALL PANEL DESIGN CHART

MATERIAL PROPERTIES

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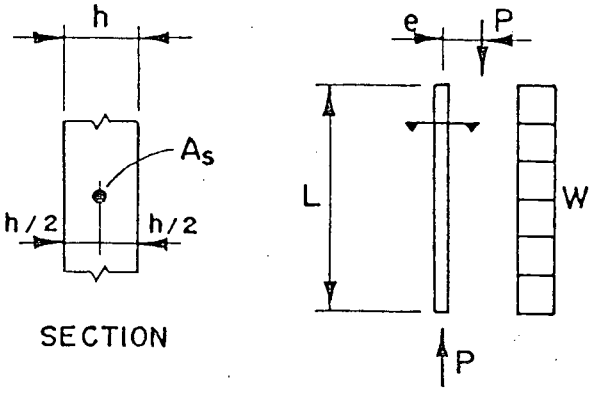
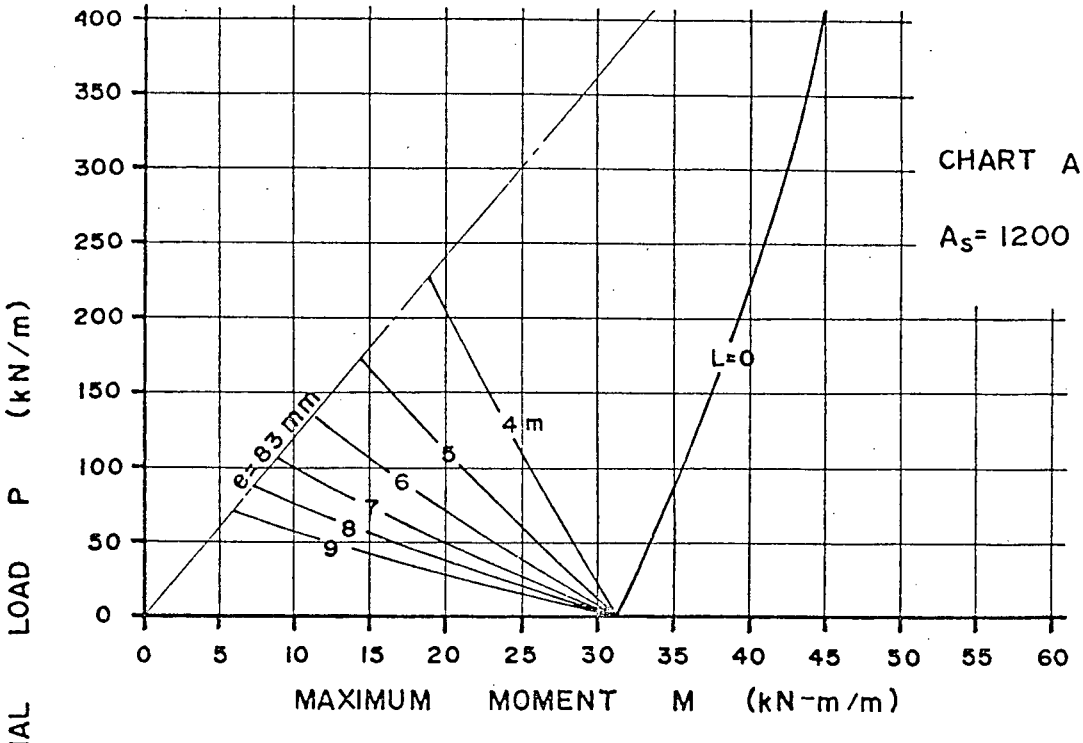
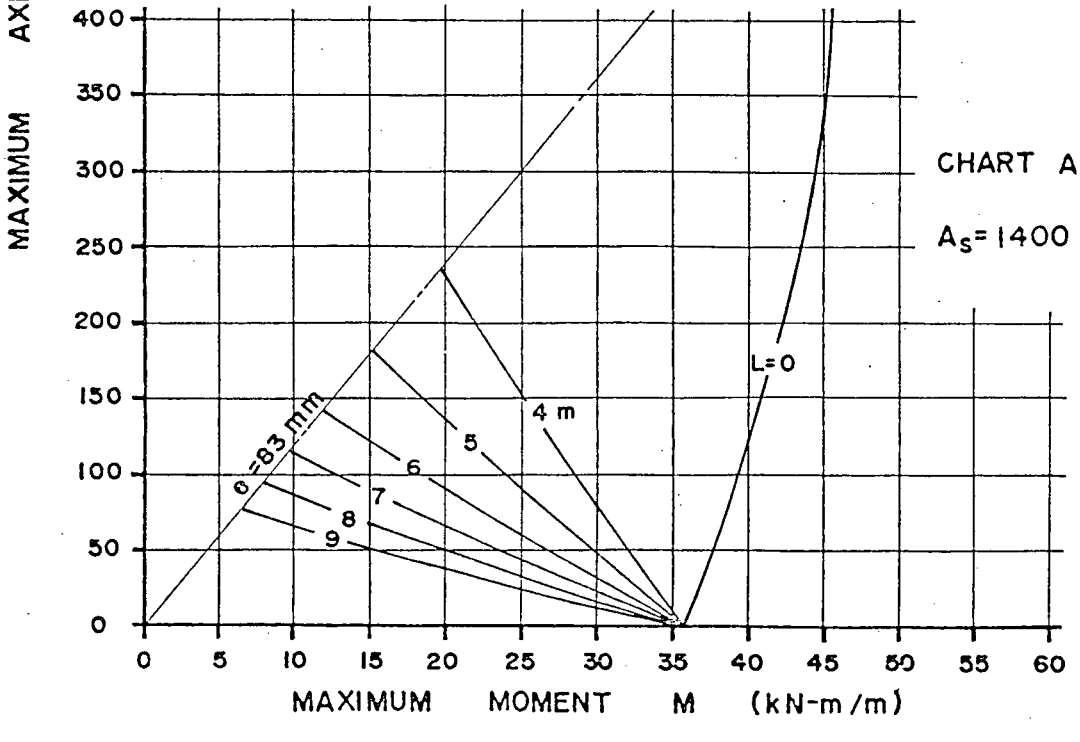
$f_y = 400$ MPa

$h = 165$ mm







CONCRETE WALL PANEL DESIGN CHART

MATERIAL PROPERTIES

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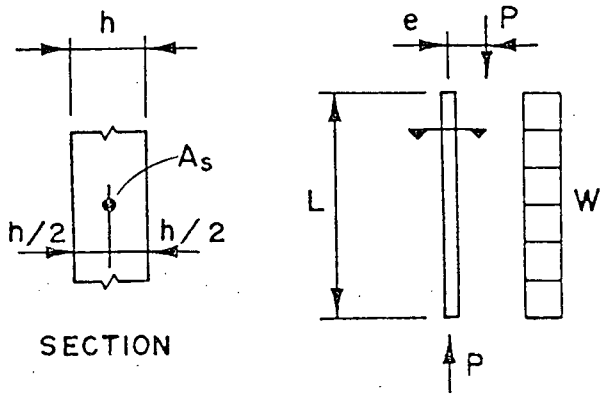
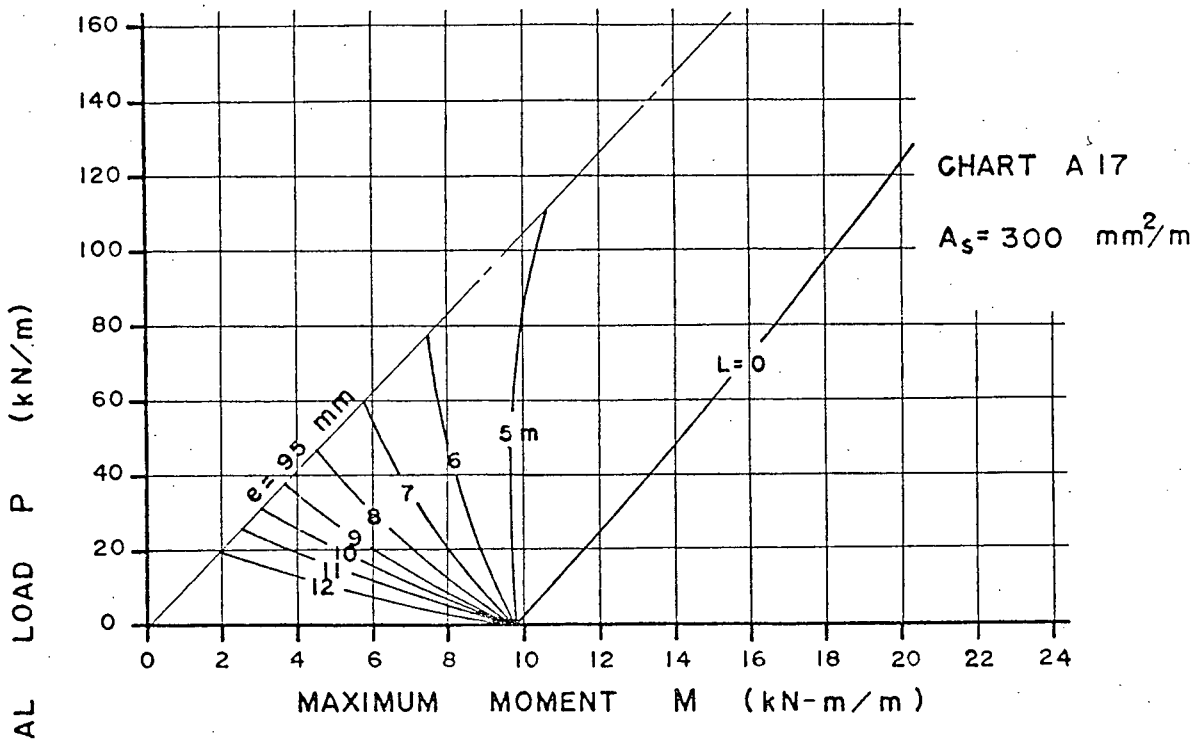
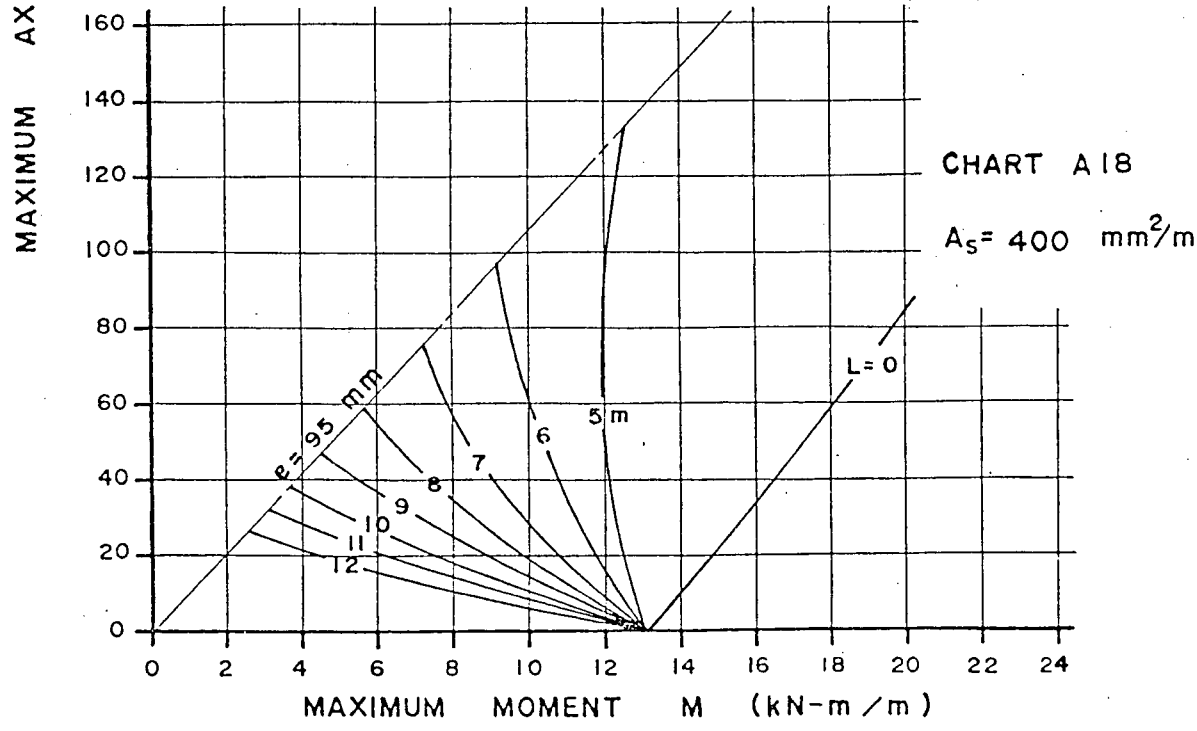
$h = 190$ mm







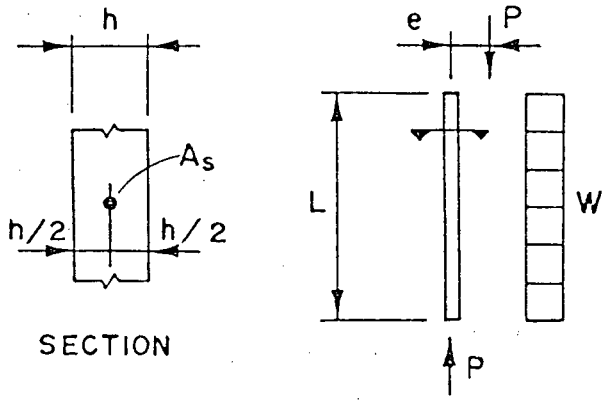
CONCRETE WALL PANEL DESIGN CHART

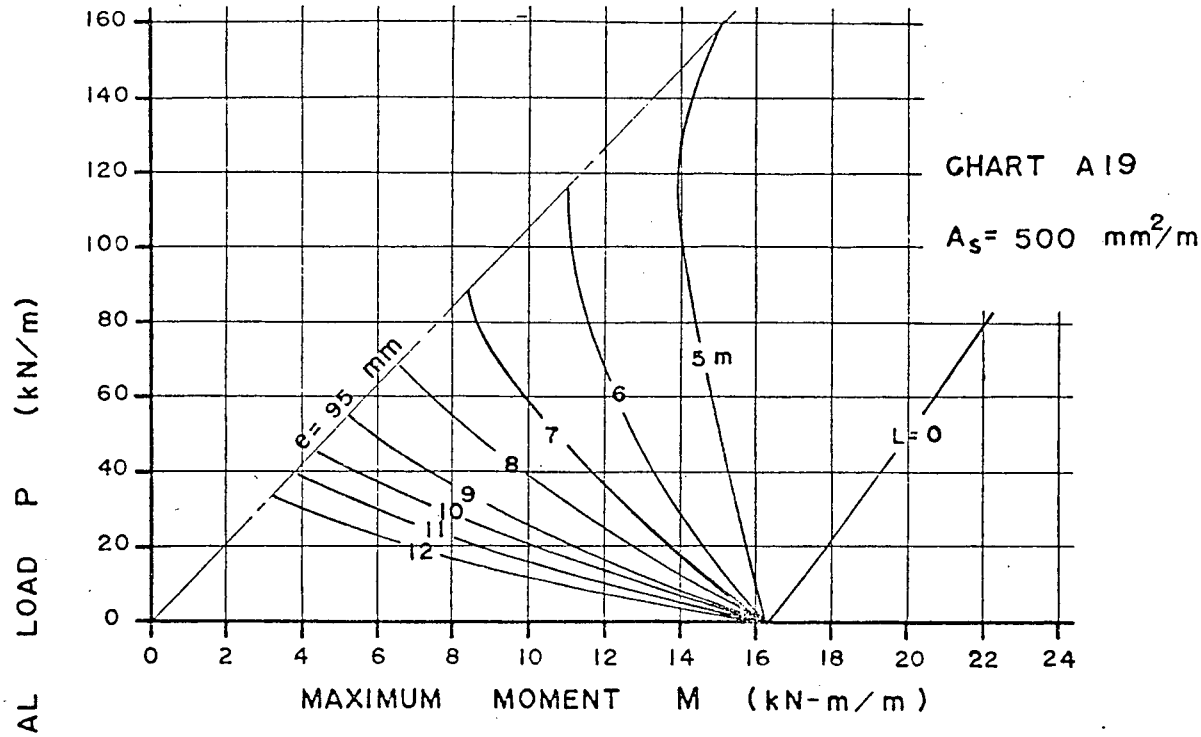
MATERIAL PROPERTIES

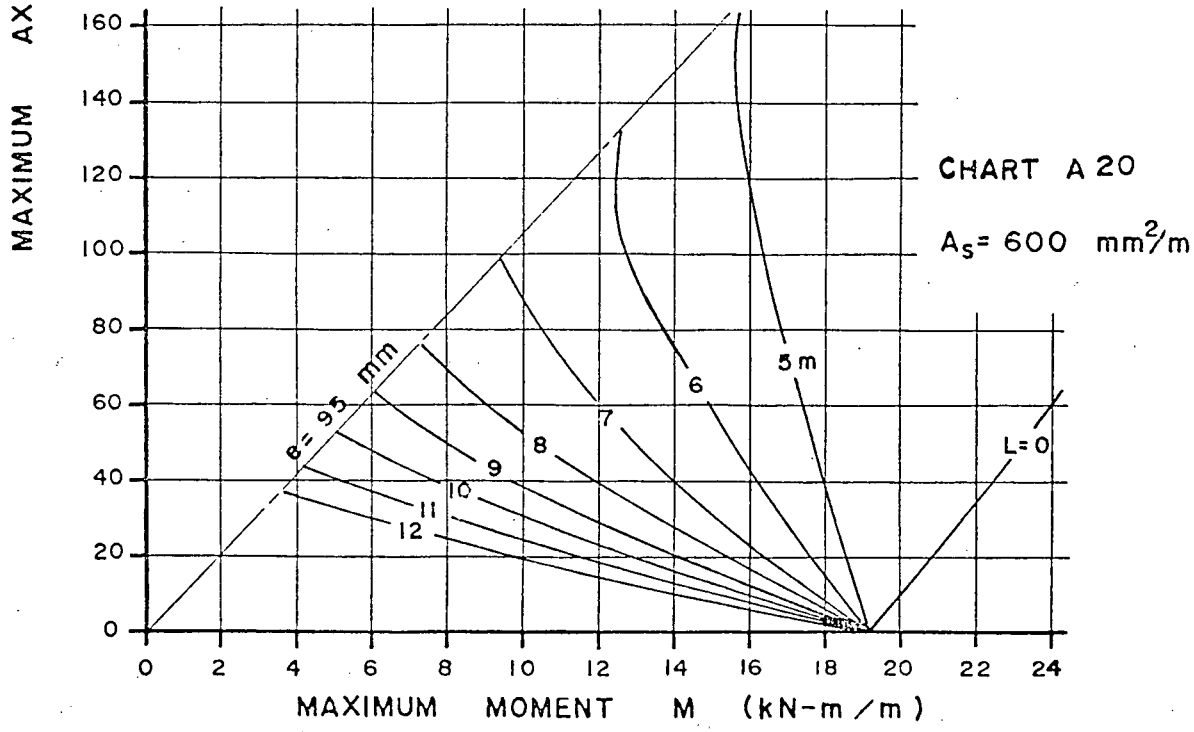
$f'_c = 25$ MPa

$f_y = 400$ MPa

$h = 190$ mm







CONCRETE WALL PANEL DESIGN CHART

MATERIAL PROPERTIES

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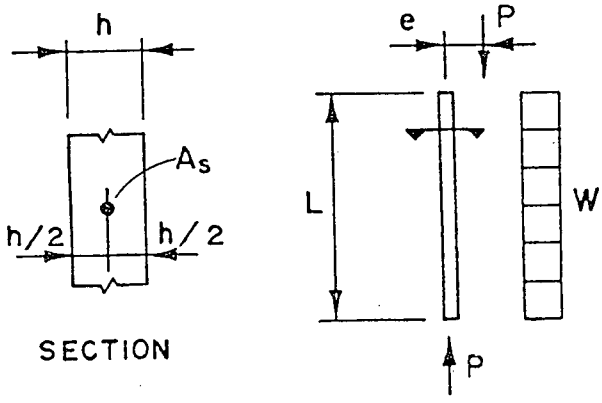
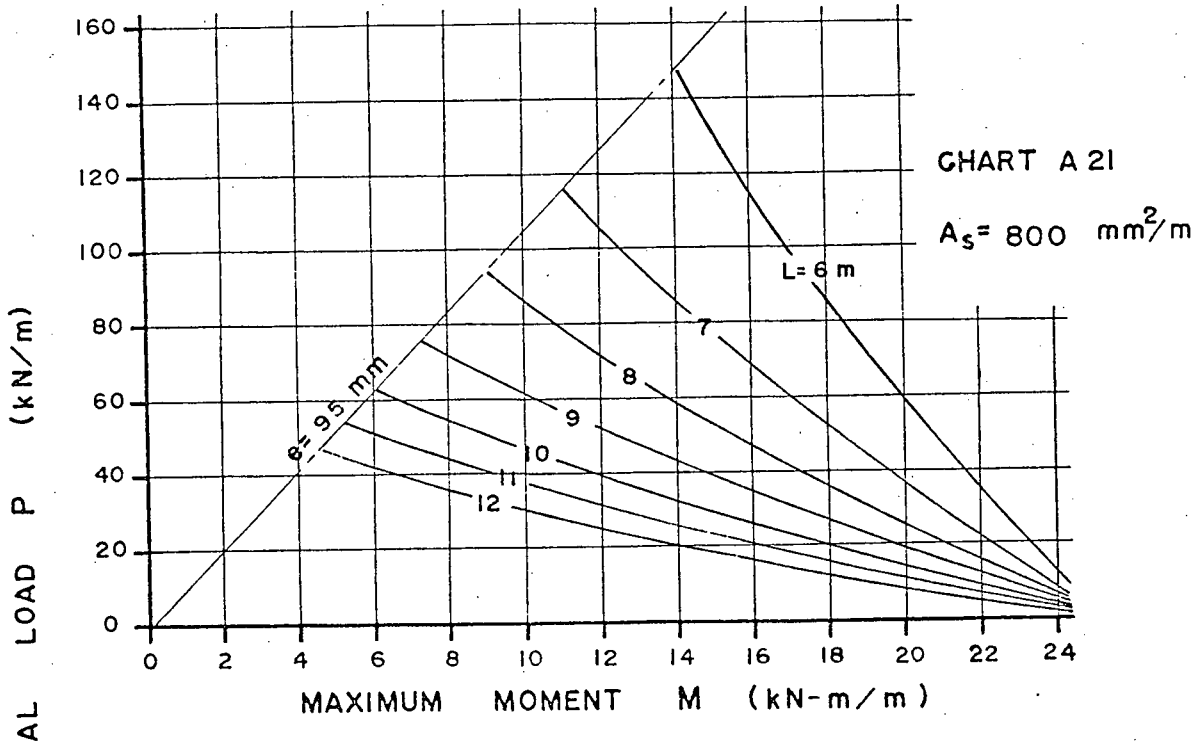
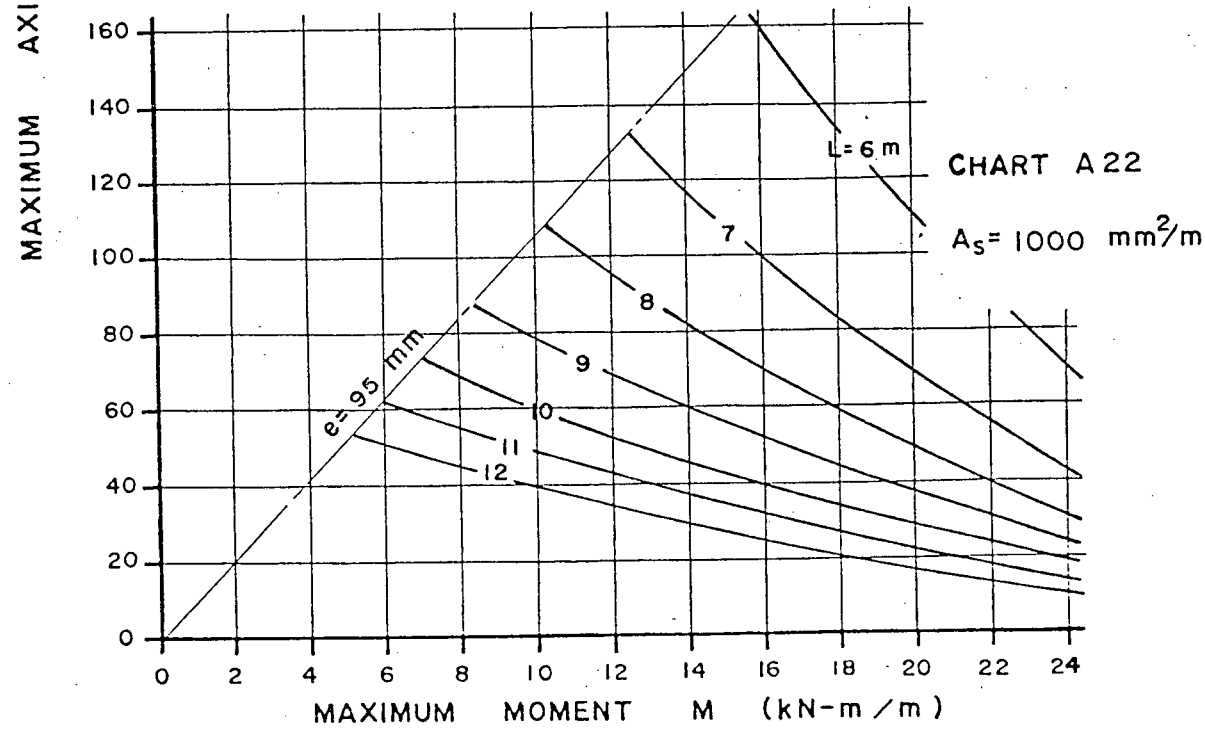
$f_y = 400$ MPa

$h = 190$ mm







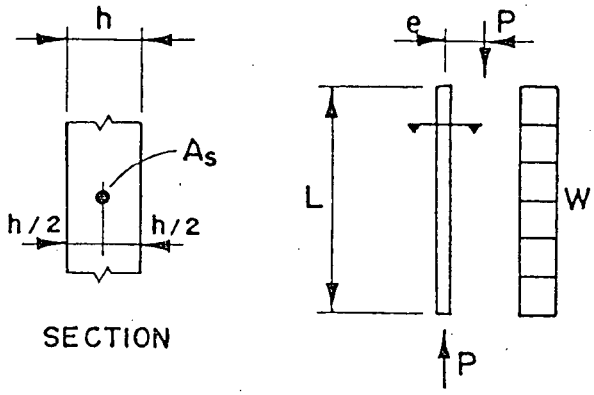
CONCRETE WALL PANEL DESIGN CHART

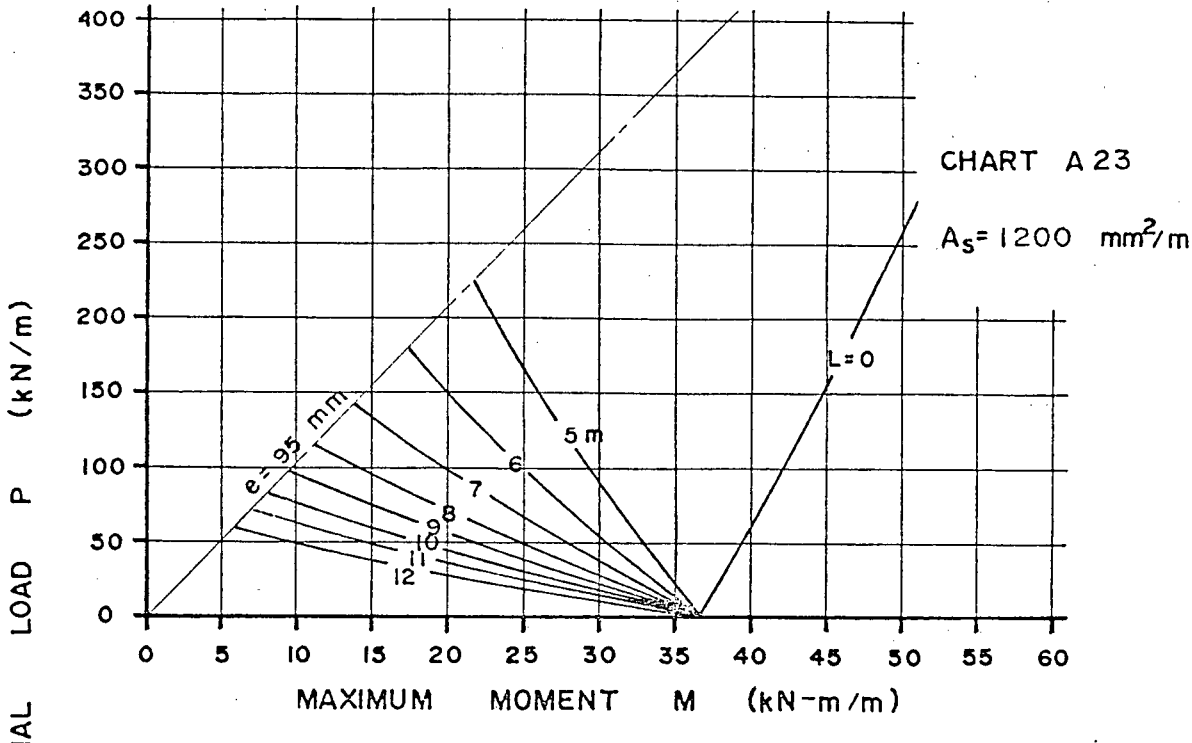
MATERIAL PROPERTIES

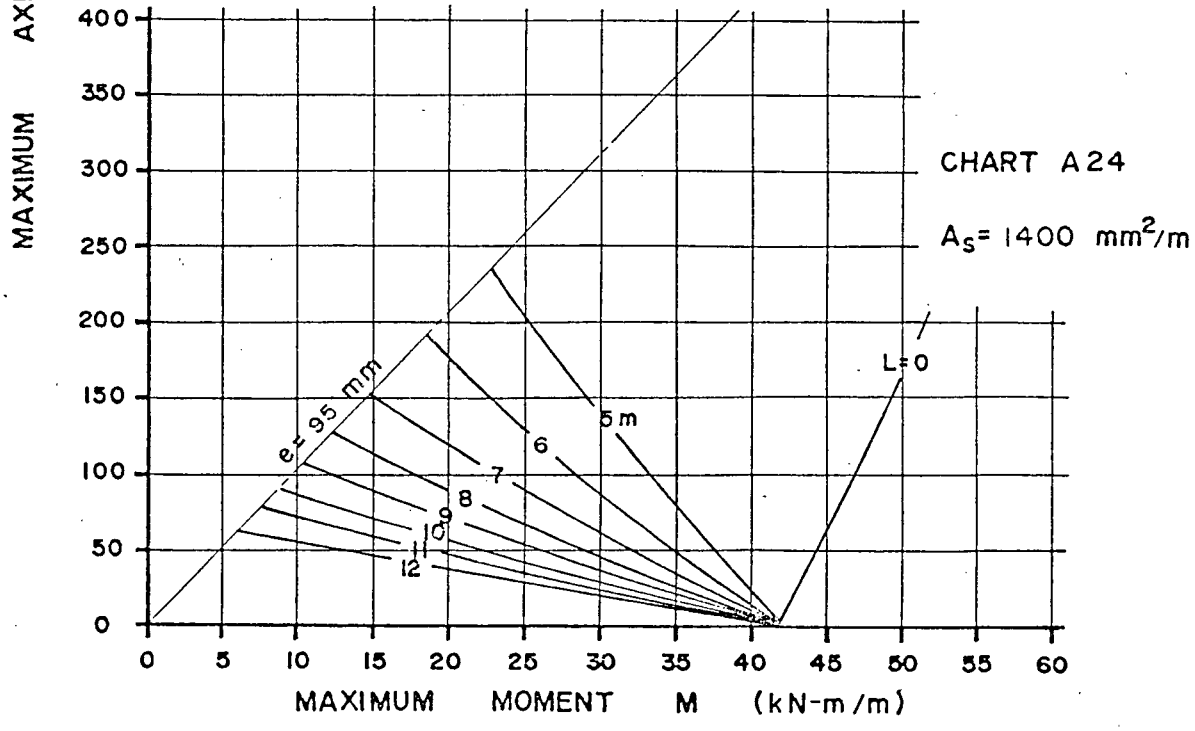
$f_c = 25$ MPa

$f_y = 400$ MPa

$h = 190$ mm







CONCRETE WALL PANEL DESIGN CHART

MATERIAL PROPERTIES

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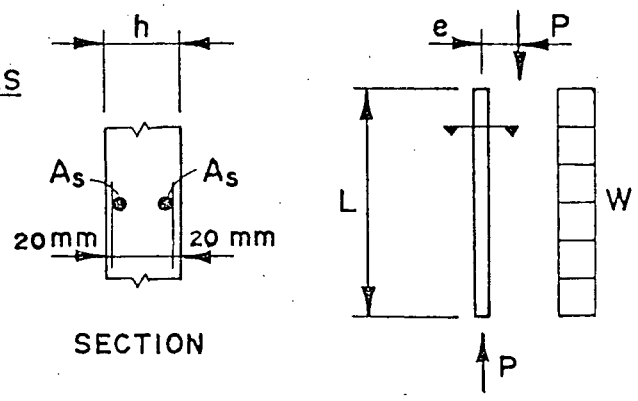
$f_s = 400$ MPa

$h = 140$ mm



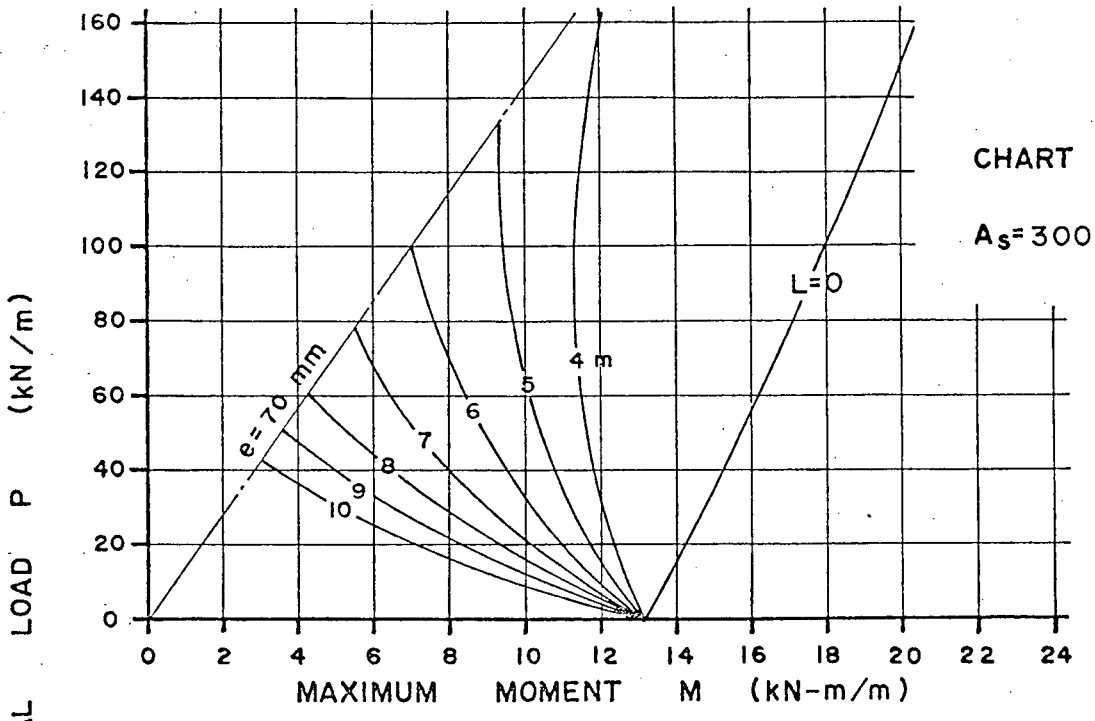


CHART B1

$A_s = 300$ mm²/m

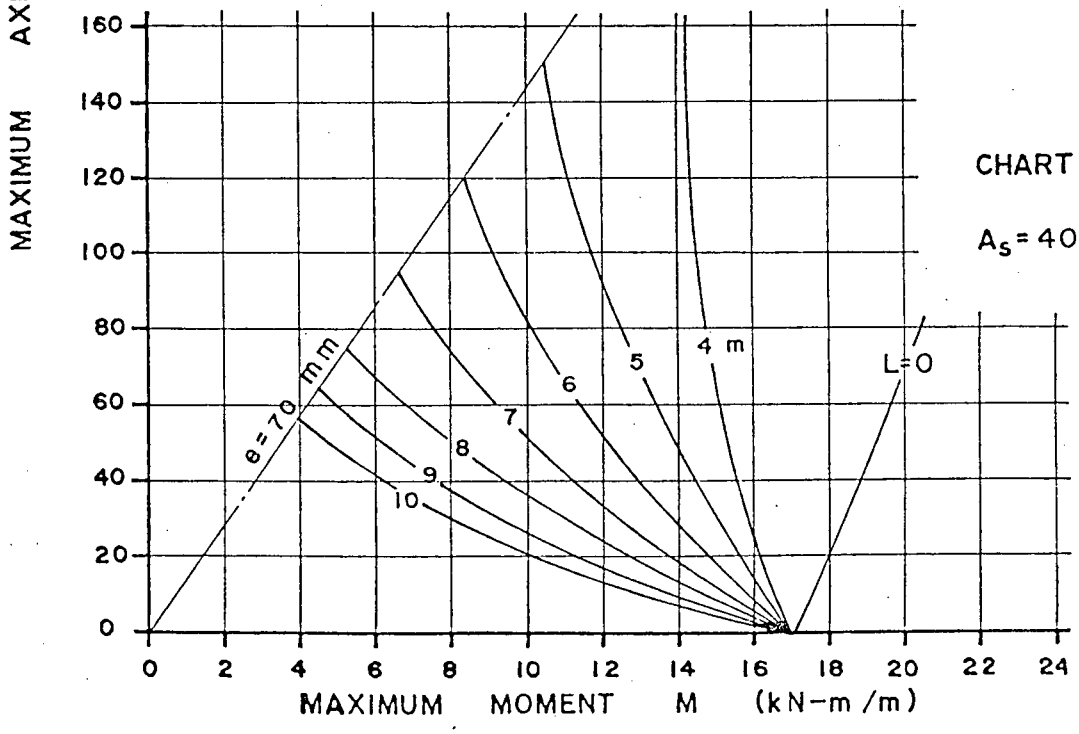


CHART B2

$A_s = 400$ mm²/m

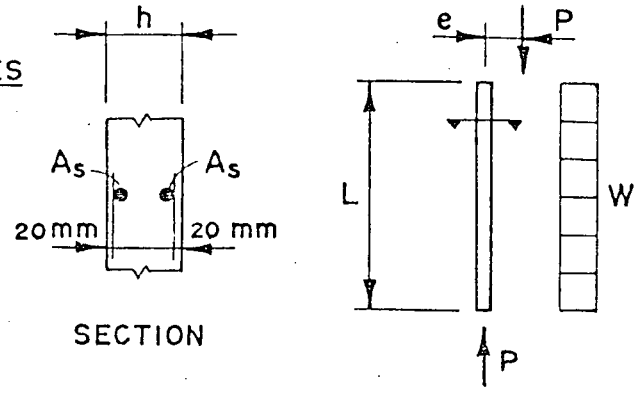
CONCRETE WALL PANEL DESIGN CHART

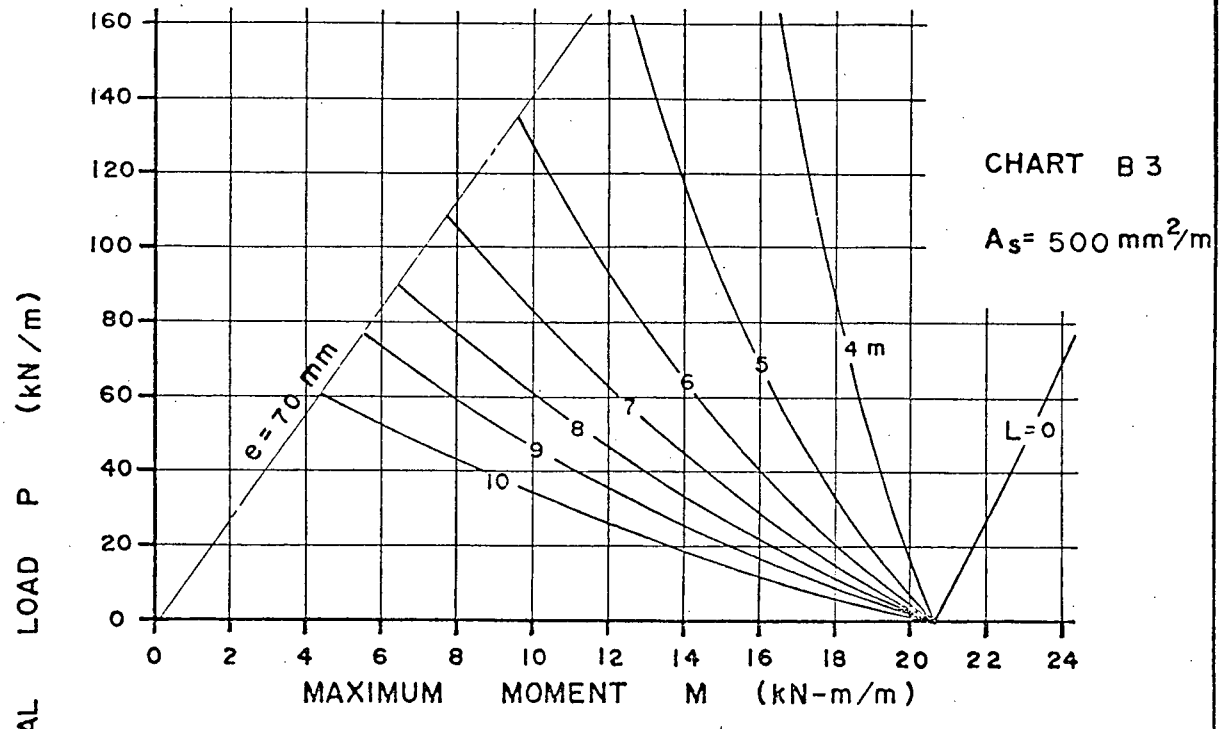
MATERIAL PROPERTIES

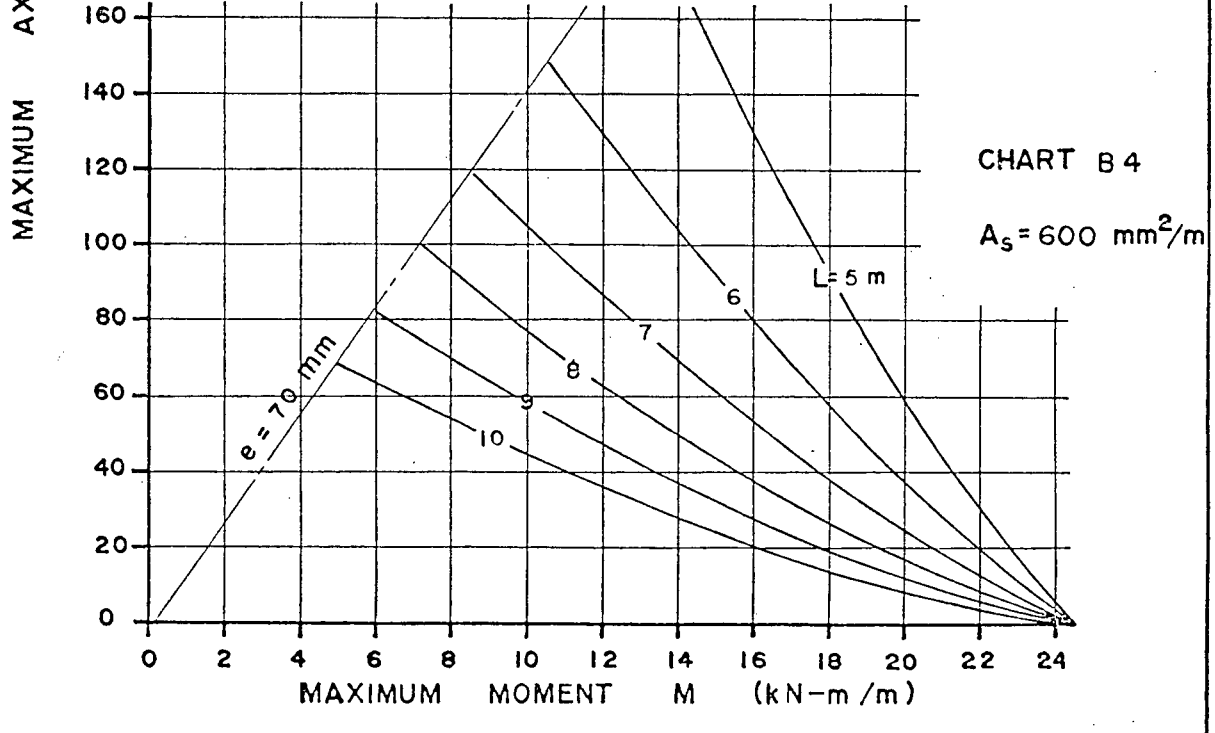
$f_c = 25$ MPa

$f_s = 400$ MPa

$h = 140$ mm







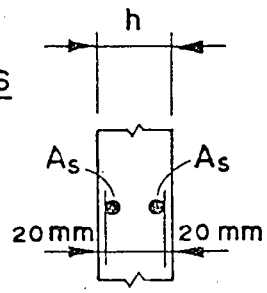
CONCRETE WALL PANEL DESIGN CHART

MATERIAL PROPERTIES

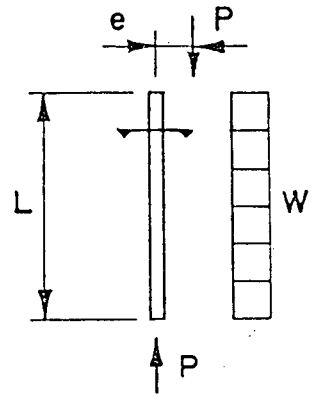
$f'_c = 25$ MPa

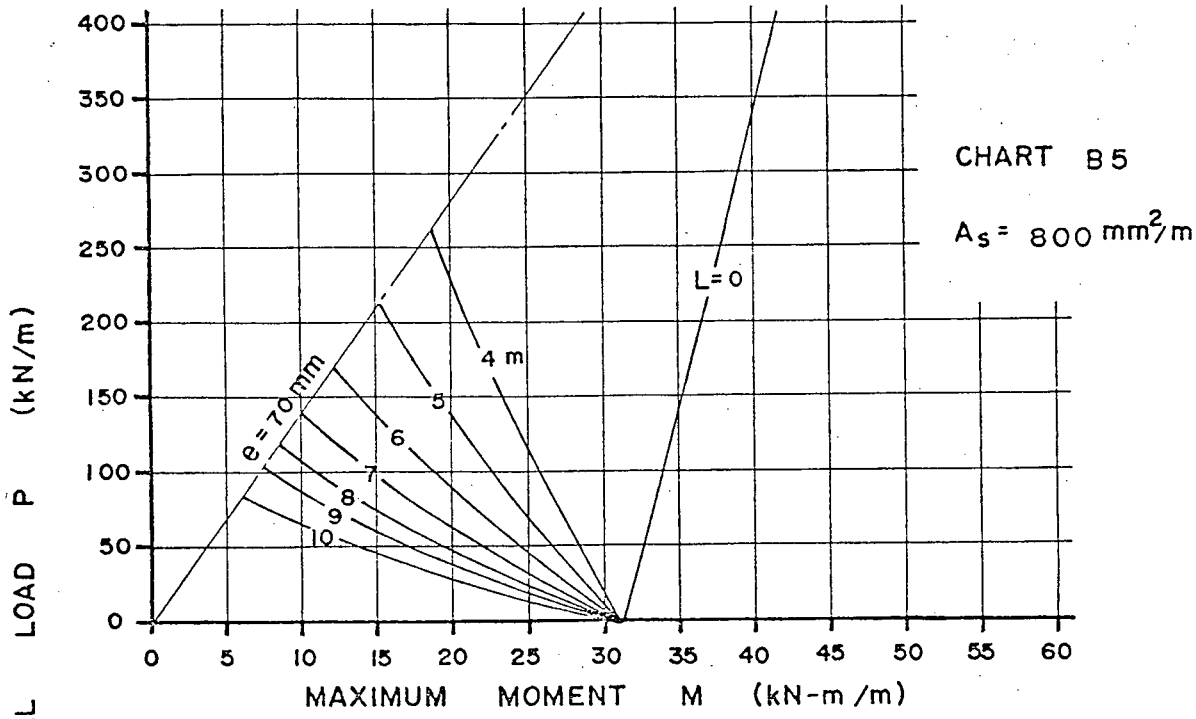
$f_y = 400$ MPa

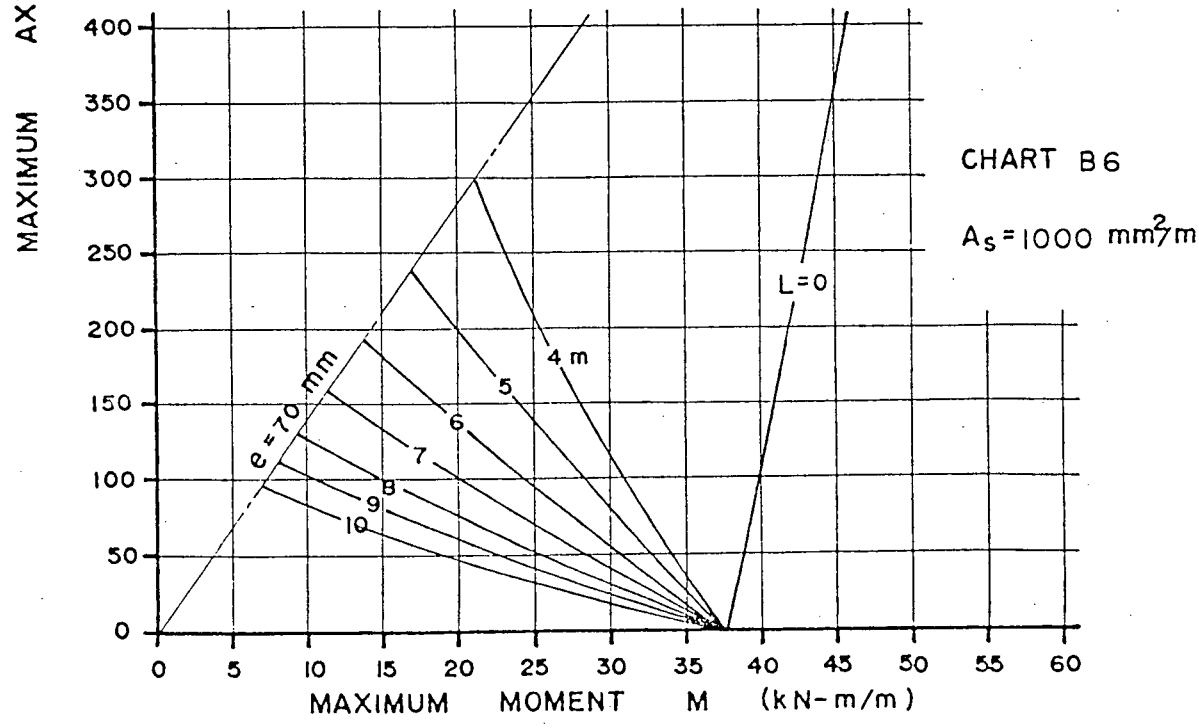
$h = 140$ mm



SECTION







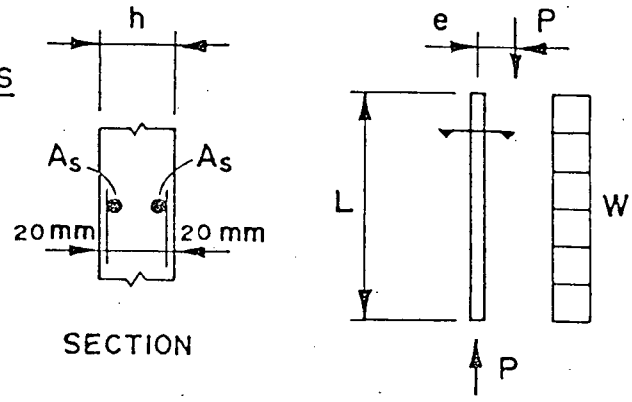
CONCRETE WALL PANEL DESIGN CHART

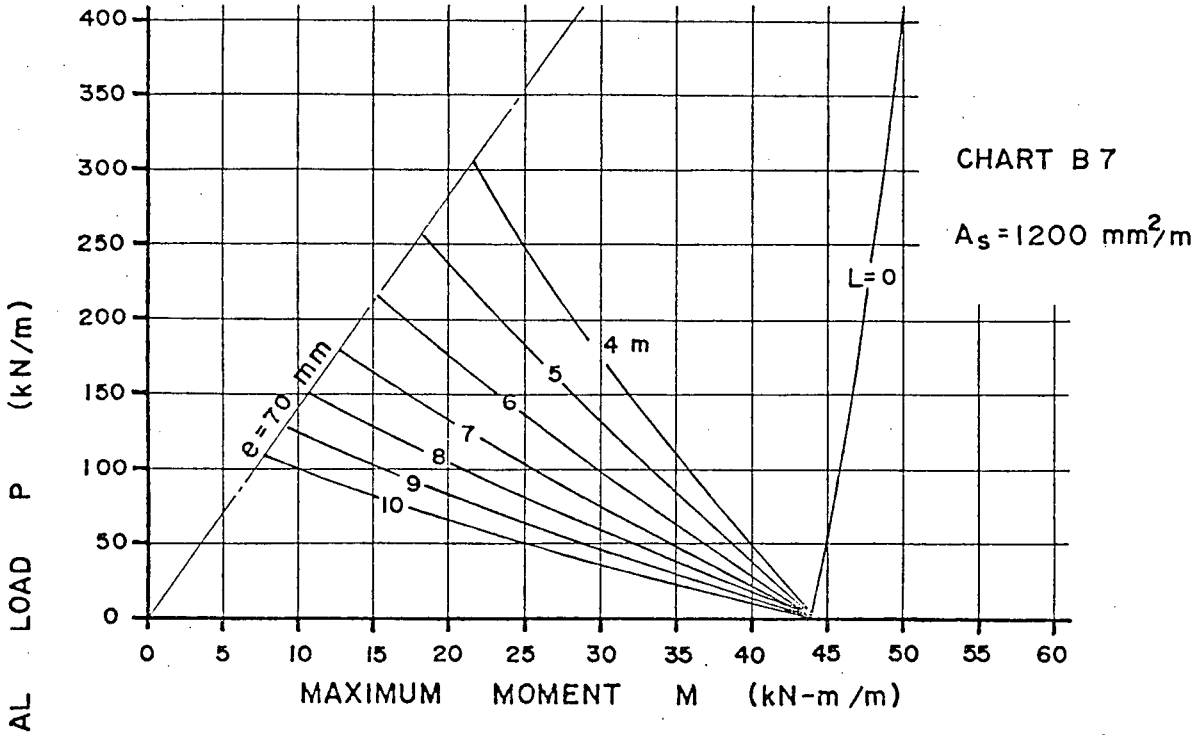
MATERIAL PROPERTIES

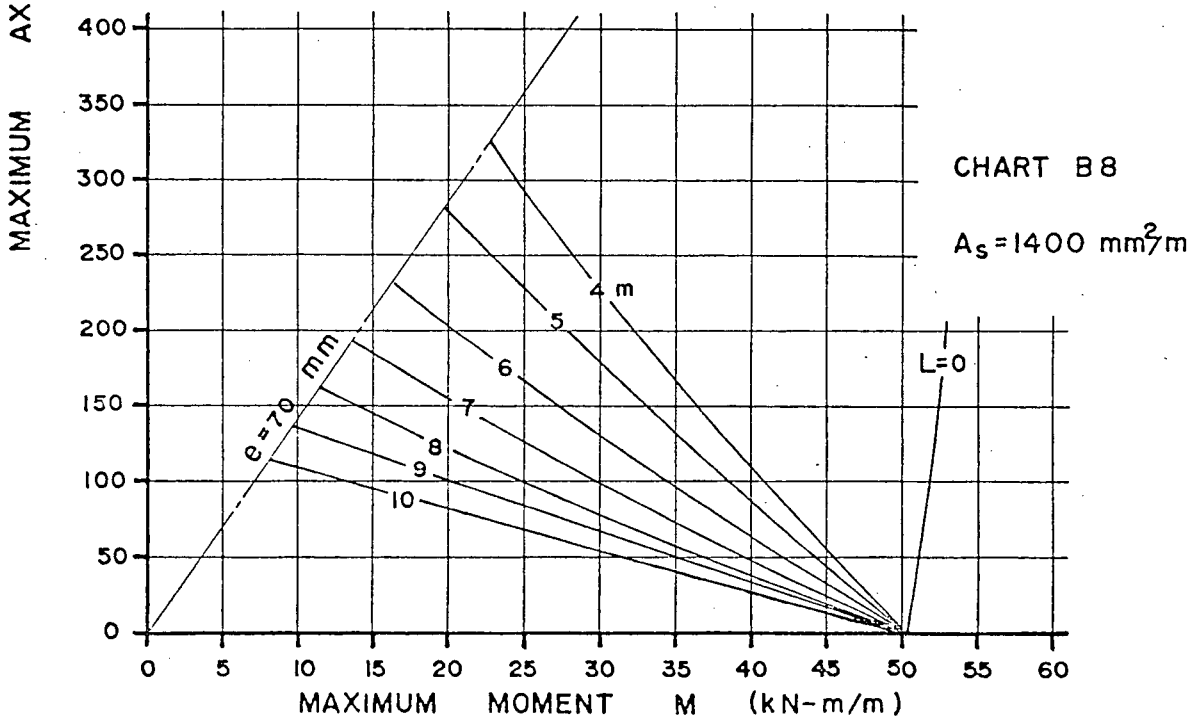
$f'_c = 25$ MPa

$f_y = 400$ MPa

$h = 140$ mm







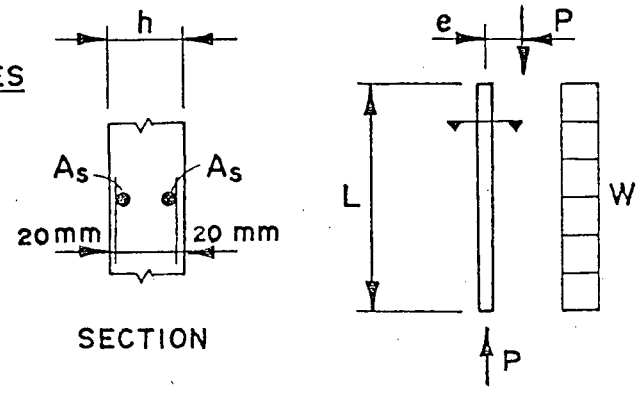
CONCRETE WALL PANEL DESIGN CHART

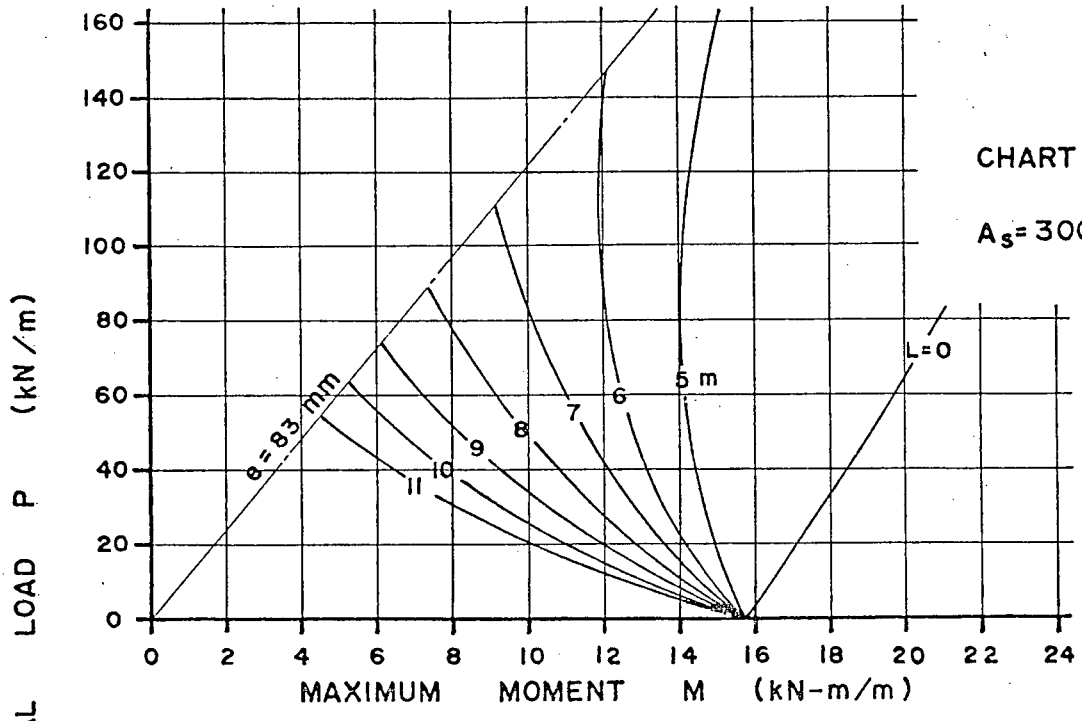
MATERIAL PROPERTIES

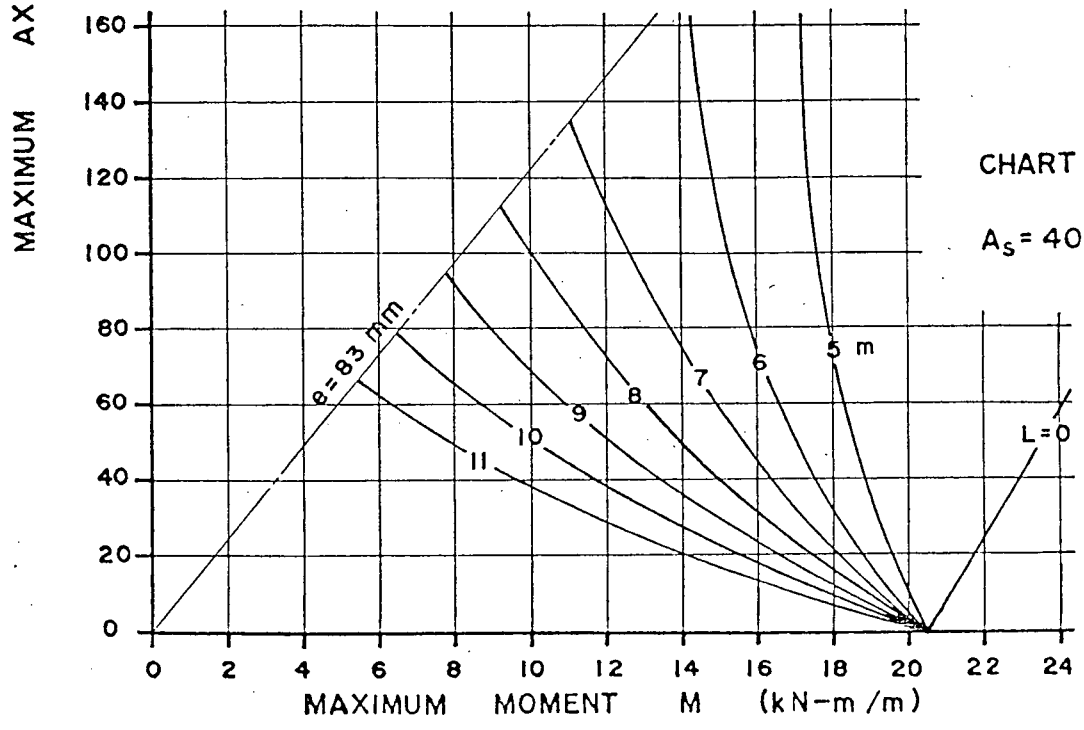
$f_c = 25$ MPa

$f_s = 400$ MPa

$h = 165$ mm







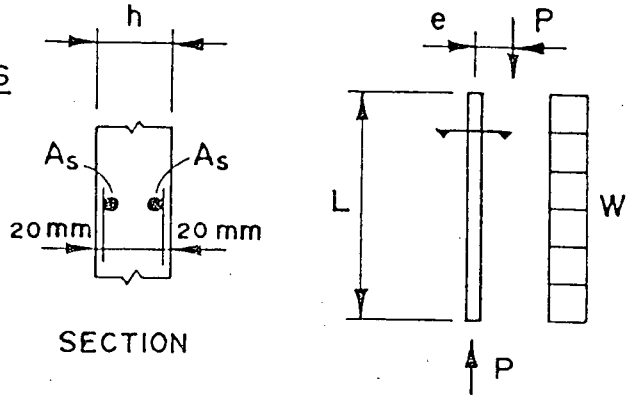
CONCRETE WALL PANEL DESIGN CHART

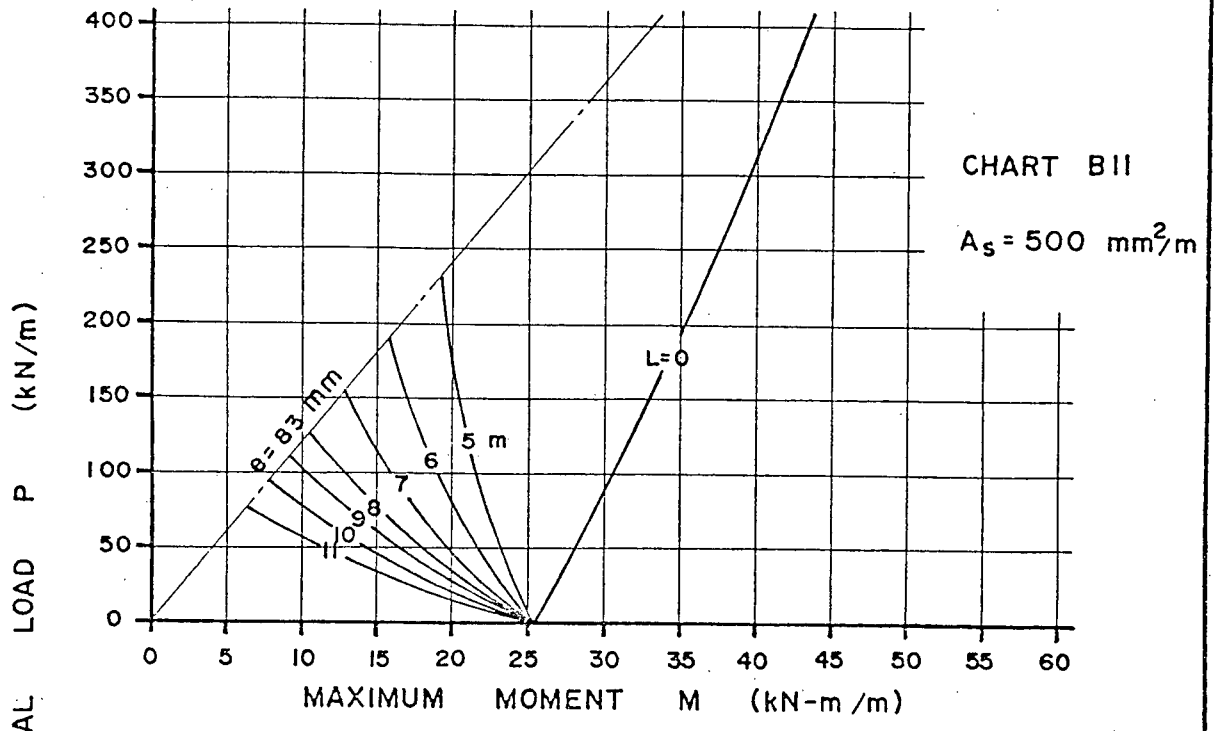
MATERIAL PROPERTIES

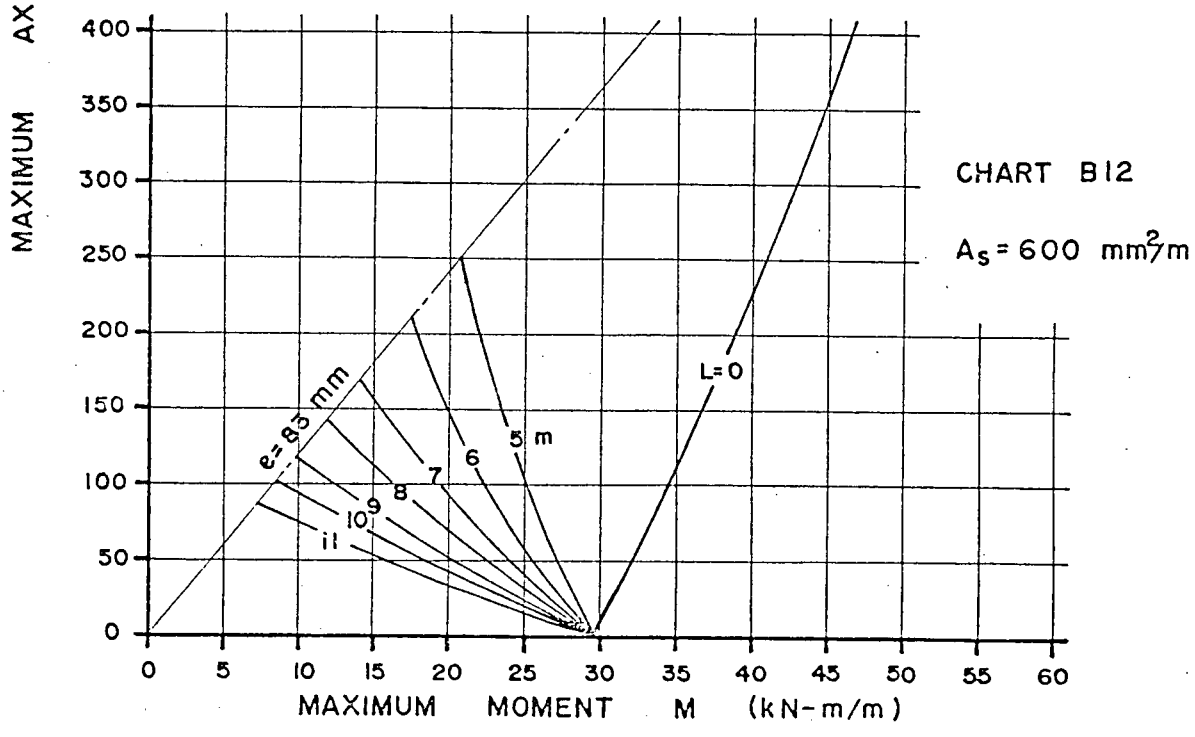
$f'_c = 25$ MPa

$f_y = 400$ MPa

$h = 165$ mm







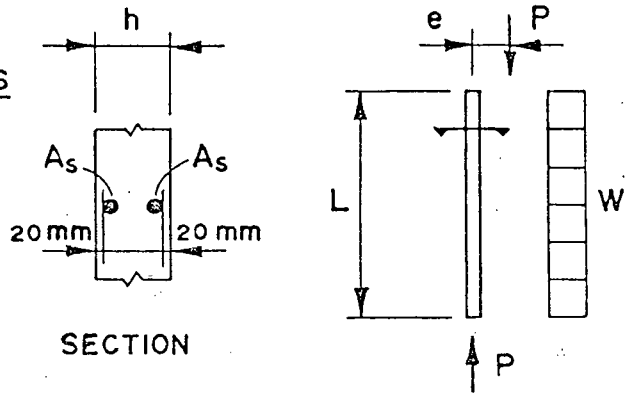
CONCRETE WALL PANEL DESIGN CHART

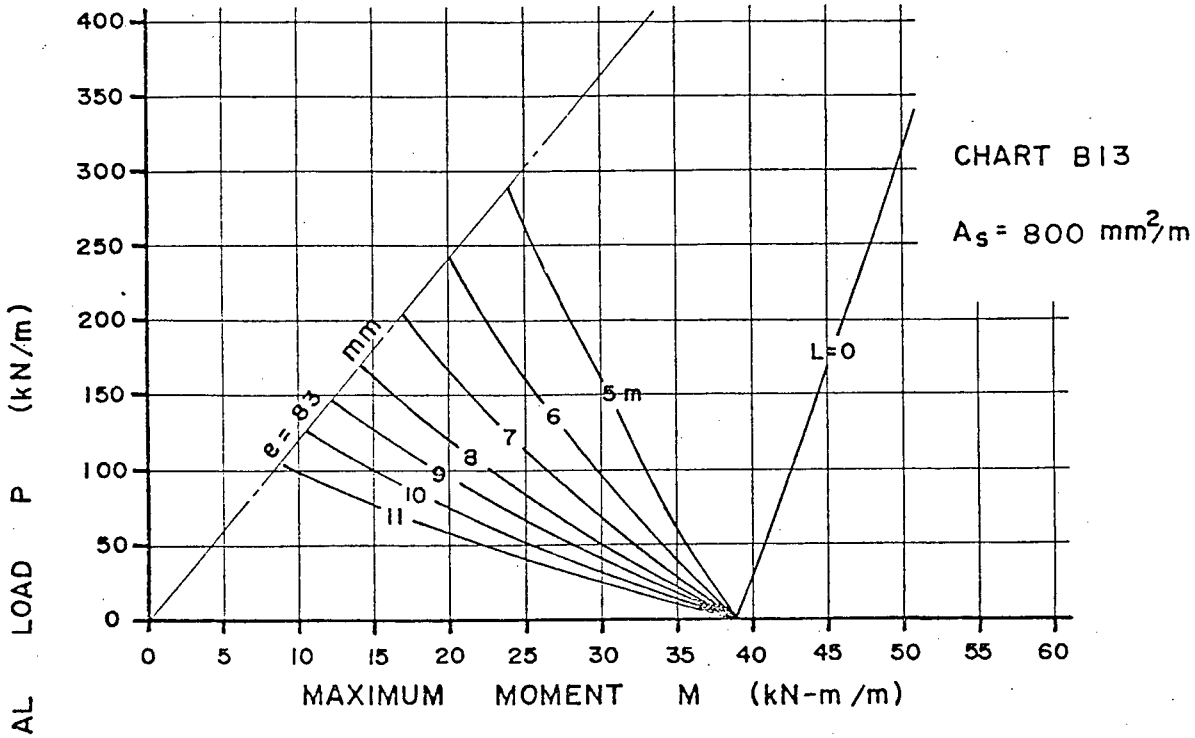
MATERIAL PROPERTIES

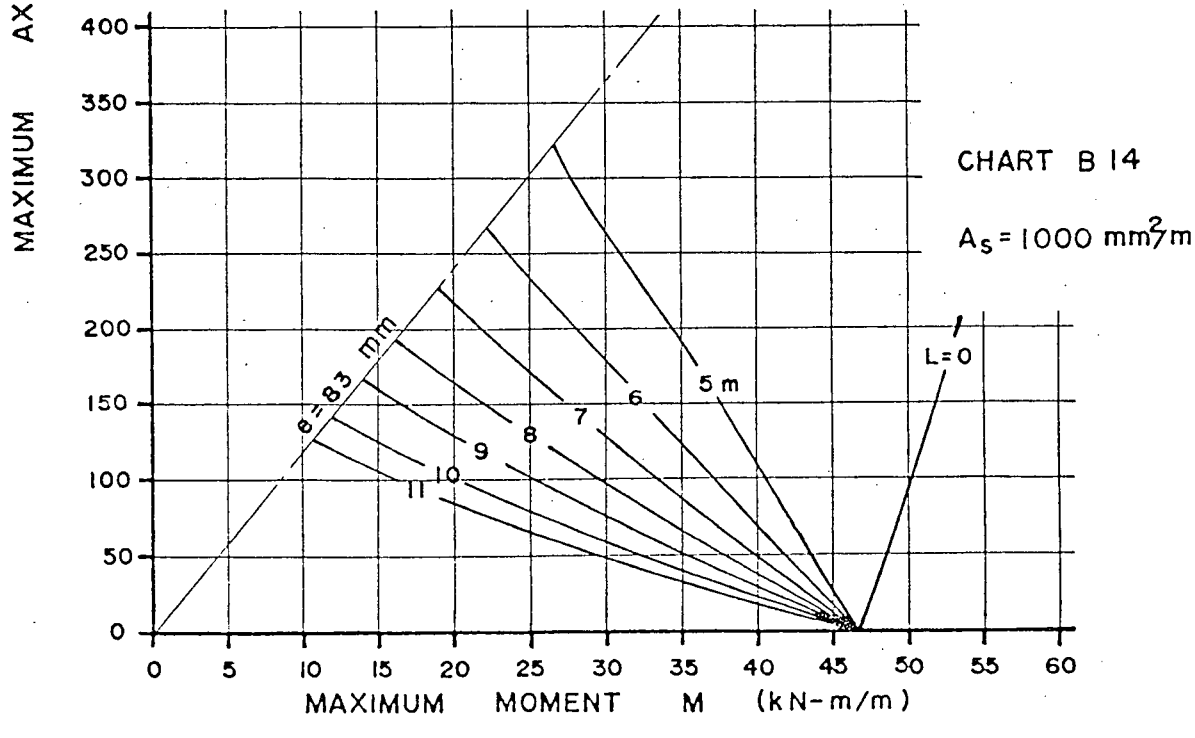
$f'_c = 25$ MPa

$f_y = 400$ MPa

$h = 165$ mm







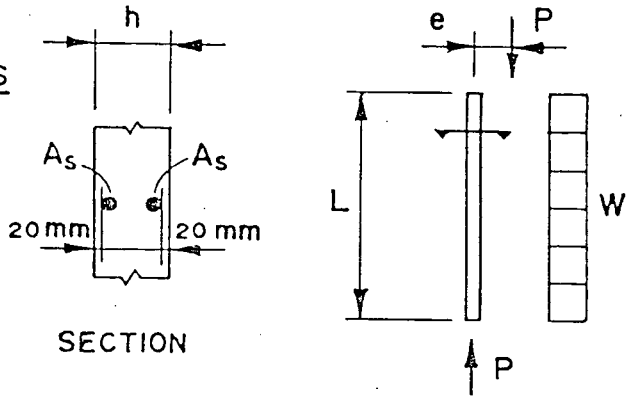
CONCRETE WALL PANEL DESIGN CHART

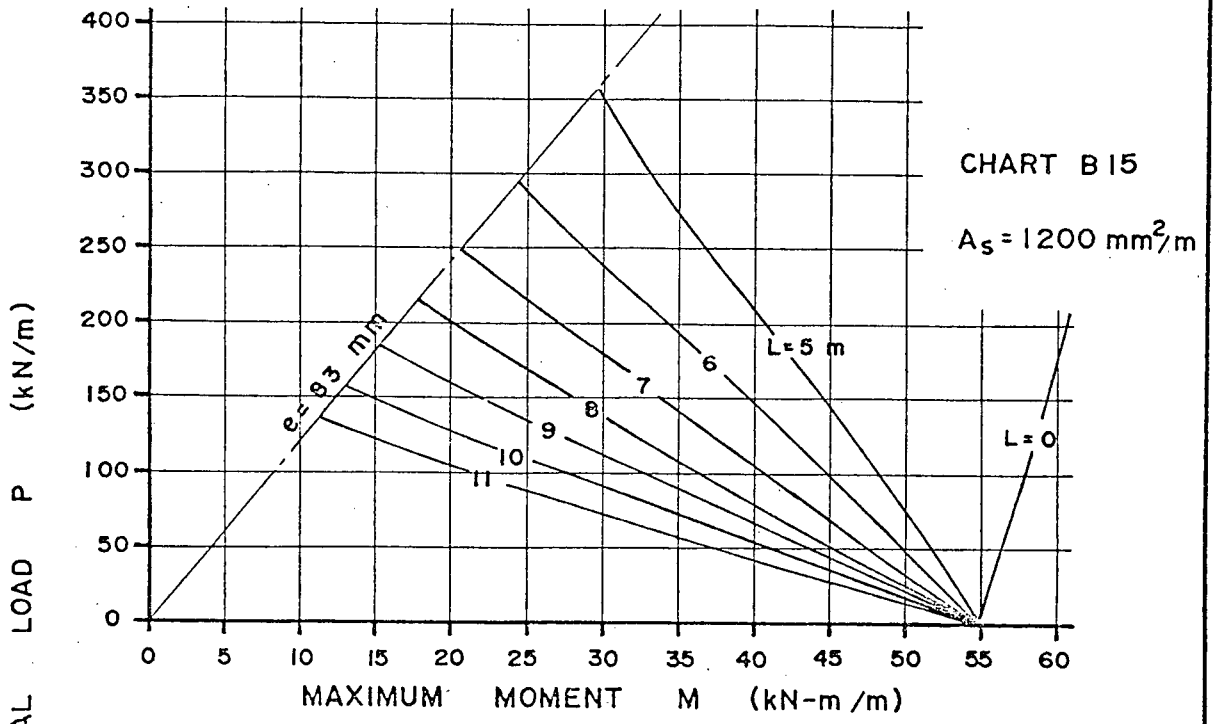
MATERIAL PROPERTIES

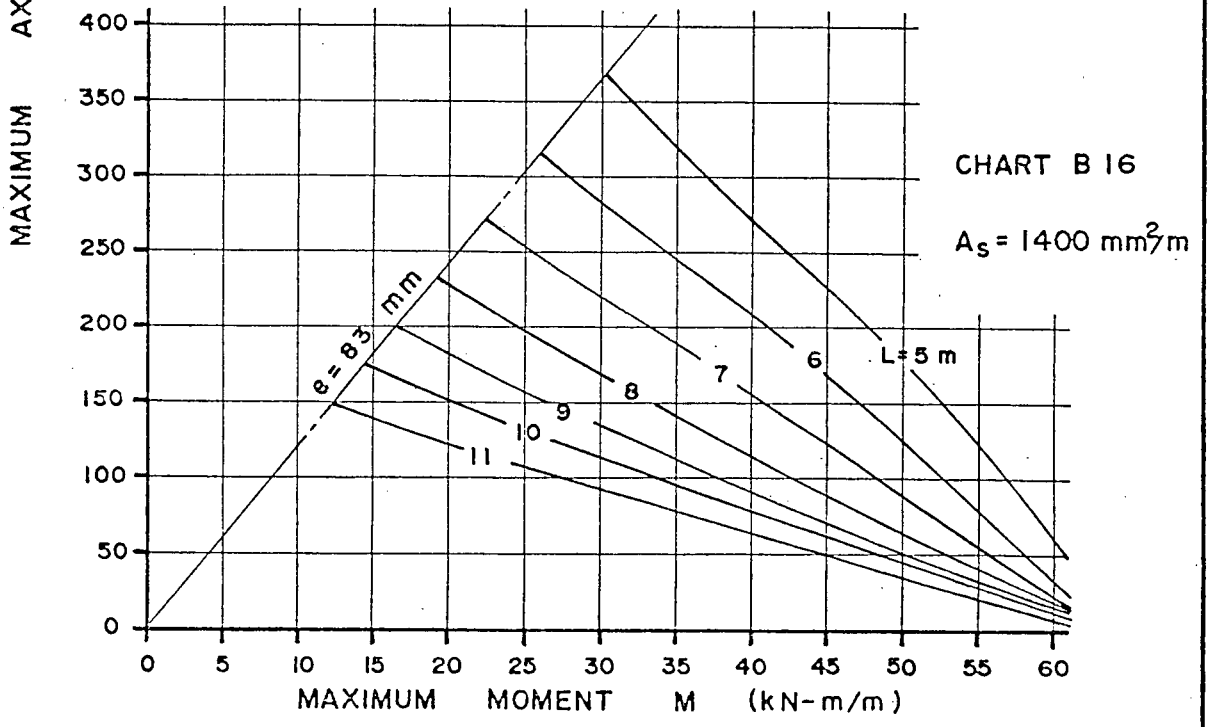
$f'_c = 25$ MPa

$f_y = 400$ MPa

$h = 165$ mm







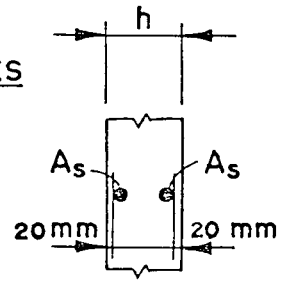
CONCRETE WALL PANEL DESIGN CHART

MATERIAL PROPERTIES

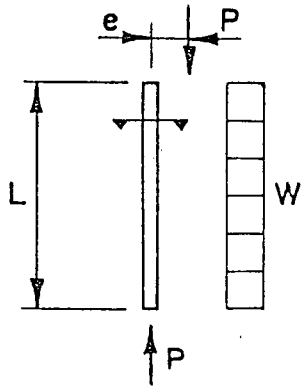
$f_c = 25$ MPa

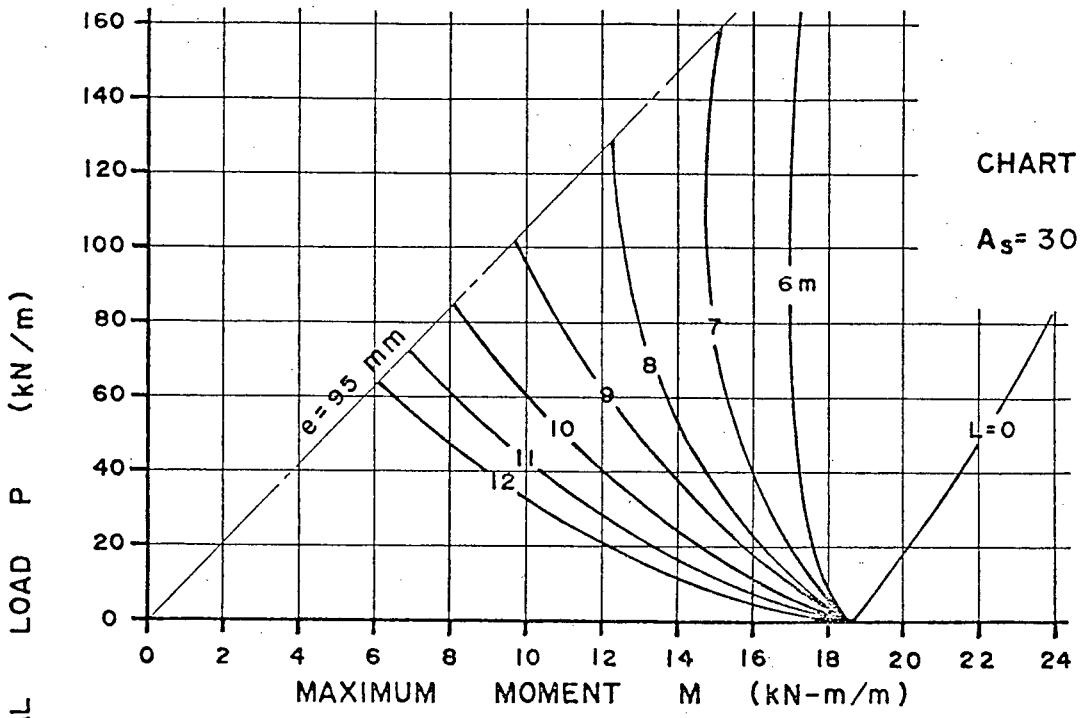
$f_s = 400$ MPa

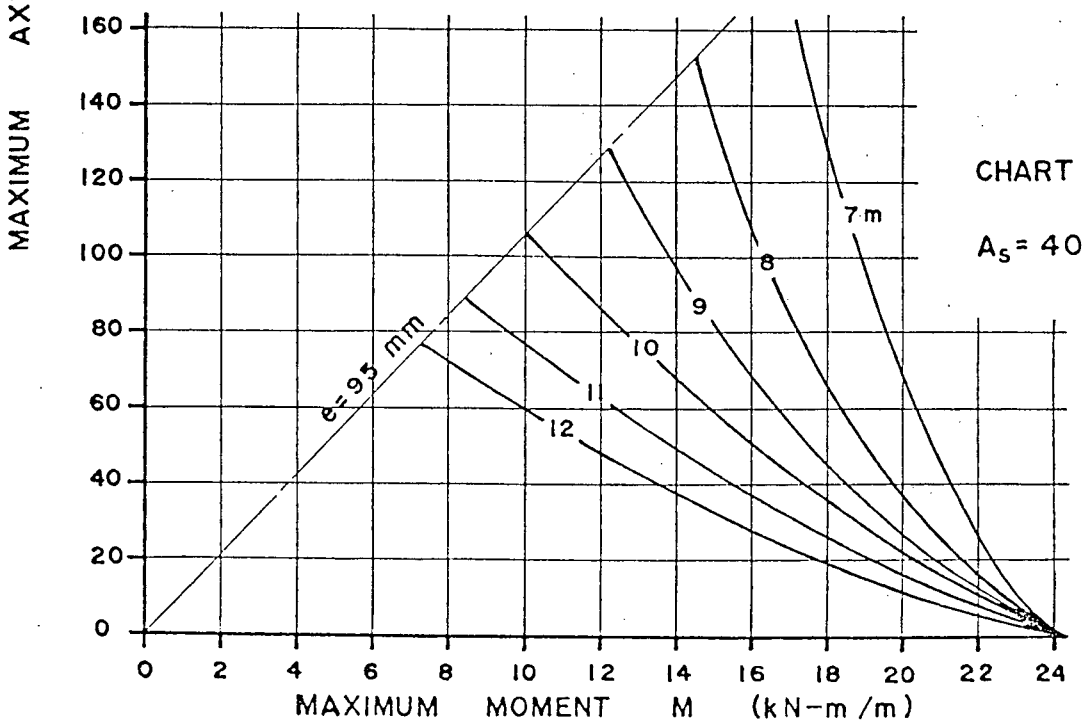
$h = 190$ mm



SECTION







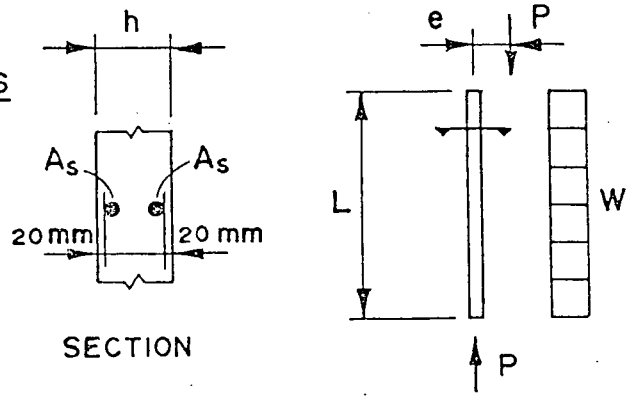
CONCRETE WALL PANEL DESIGN CHART

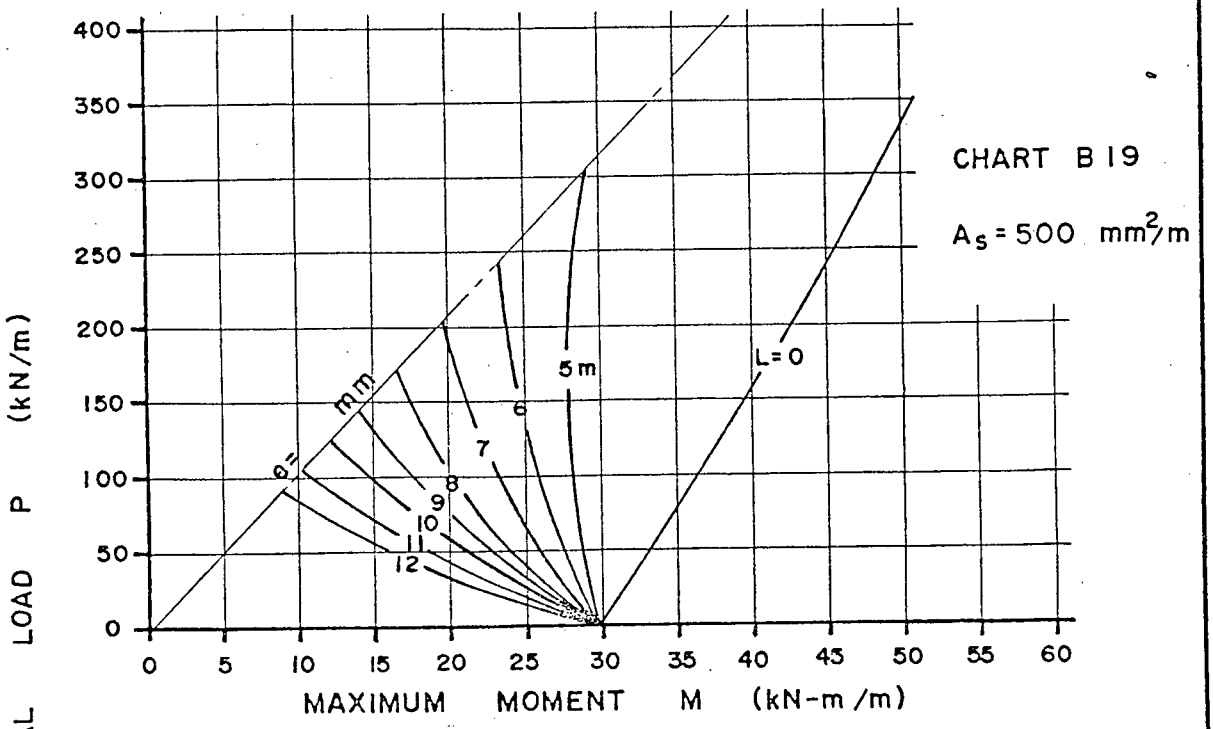
MATERIAL PROPERTIES

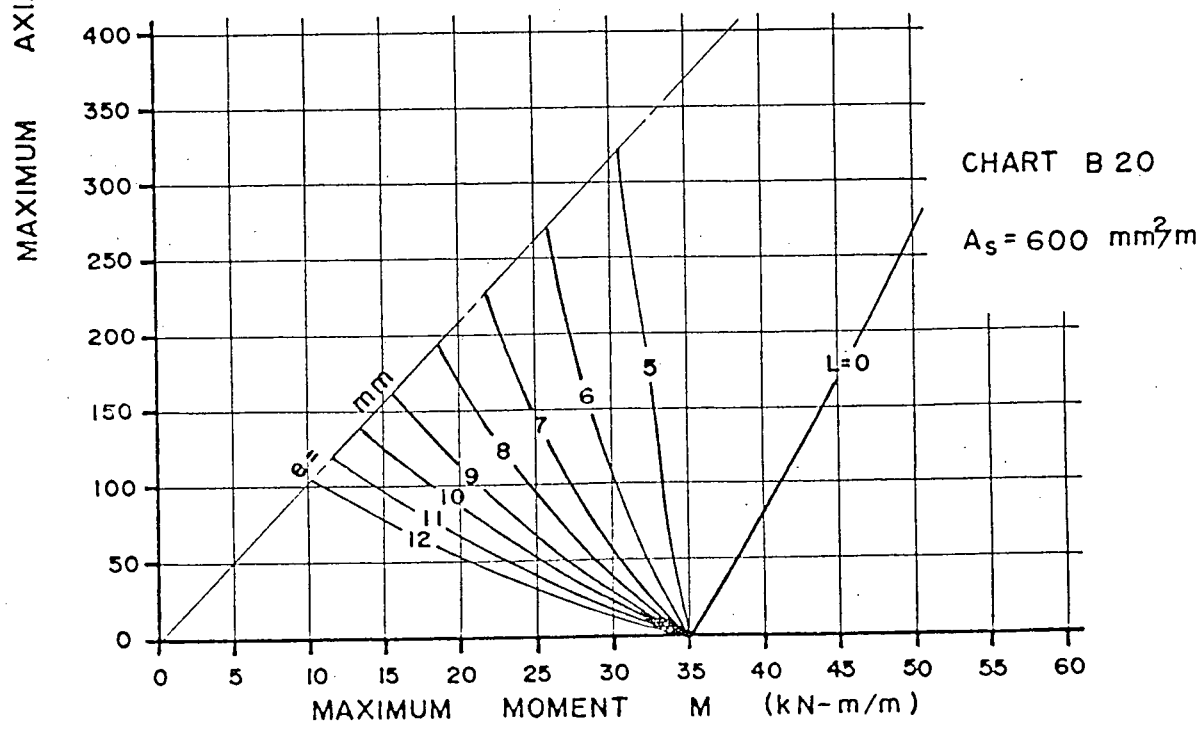
$f'_c = 25$ MPa

$f_y = 400$ MPa

$h = 190$ mm







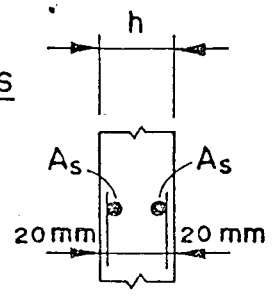
CONCRETE WALL PANEL DESIGN CHART

MATERIAL PROPERTIES

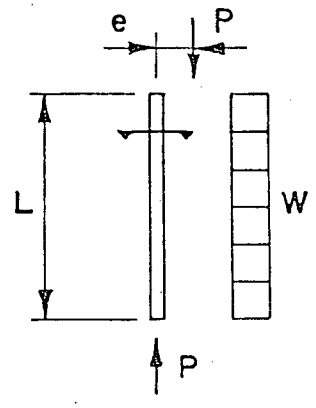
$f'_c = 25$ MPa

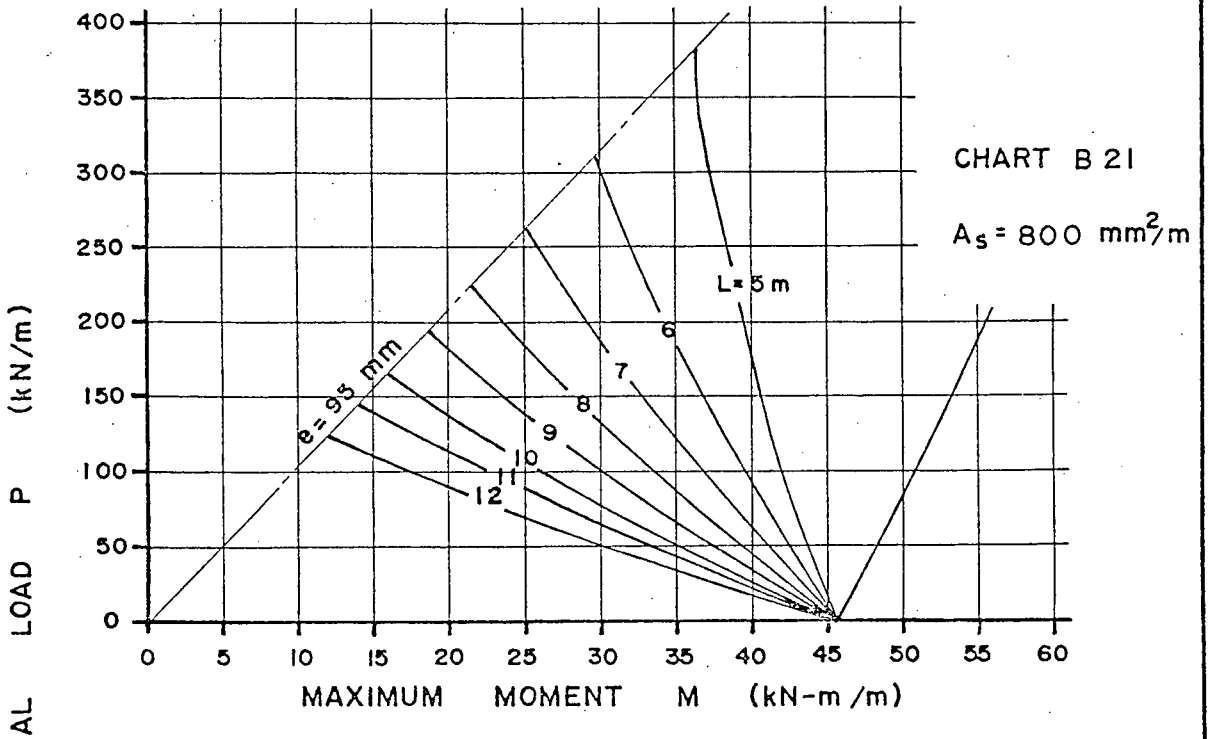
$f_y = 400$ MPa

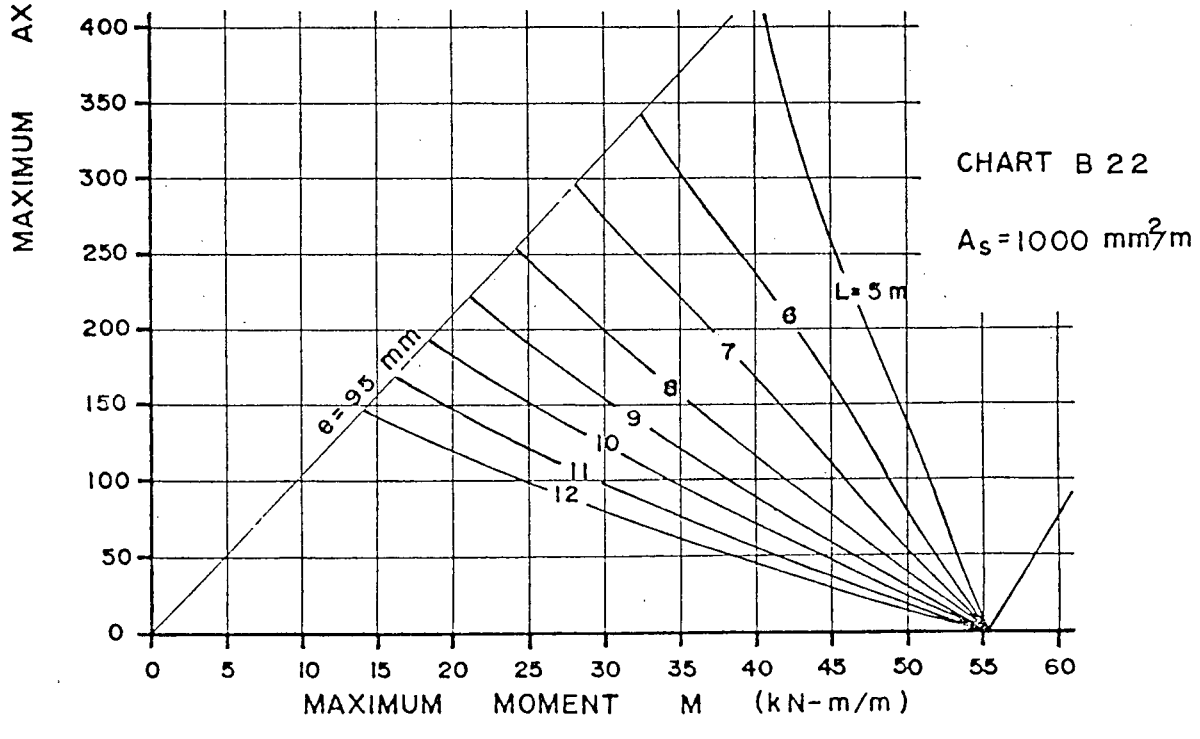
$h = 190$ mm



SECTION







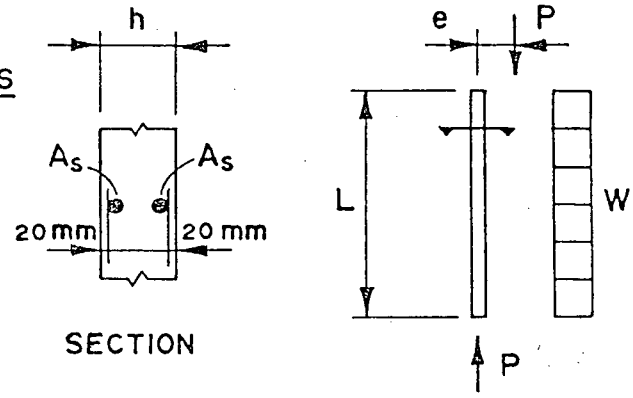
CONCRETE WALL PANEL DESIGN CHART

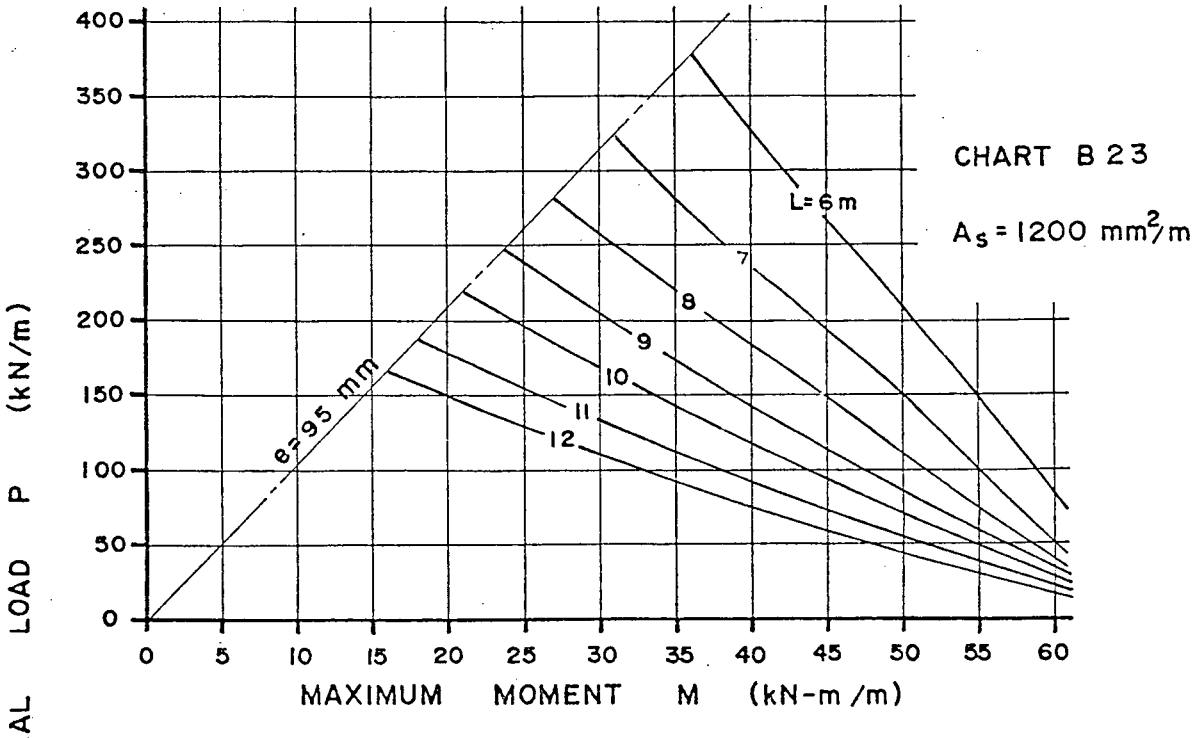
MATERIAL PROPERTIES

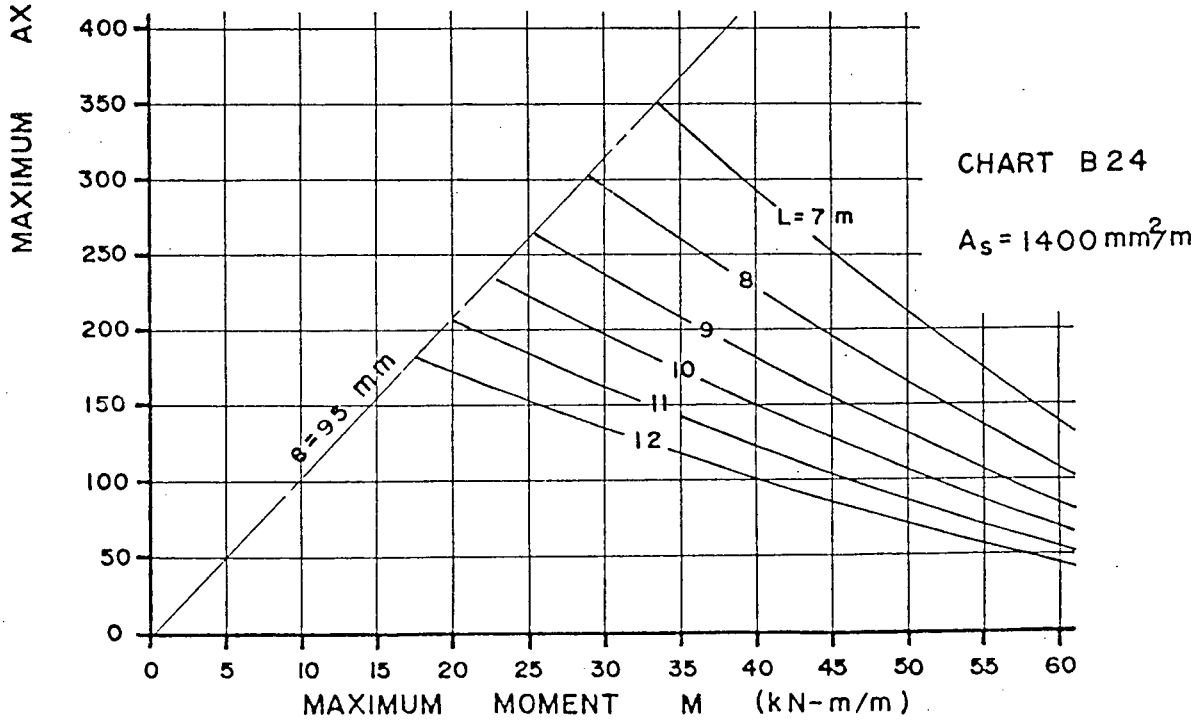
$f'_c = 25$ MPa

$f_y = 400$ MPa

$h = 190$ mm







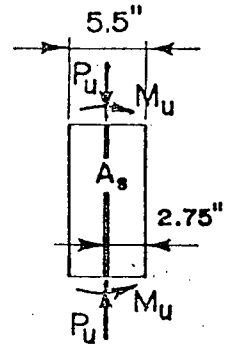
APPENDIX C

AVERAGE STIFFNESS CHARTS

CONCRETE WALL PANEL
SECTION STIFFNESS PROPERTIES

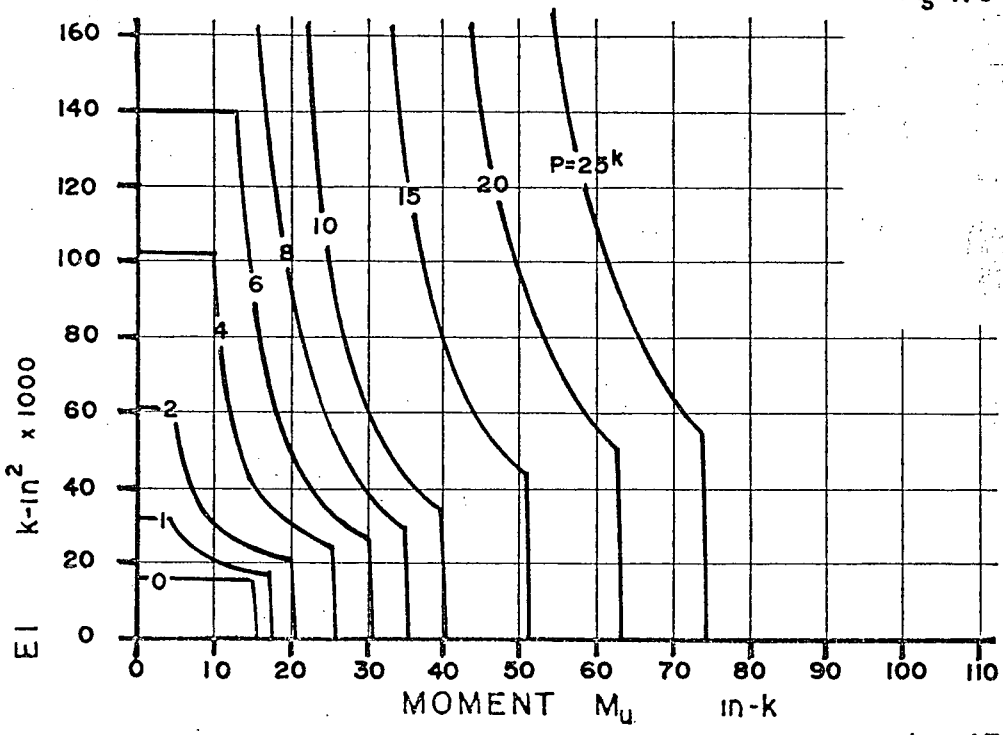
$f'_c = 4000$ PSI

$f_y = 60000$ PSI

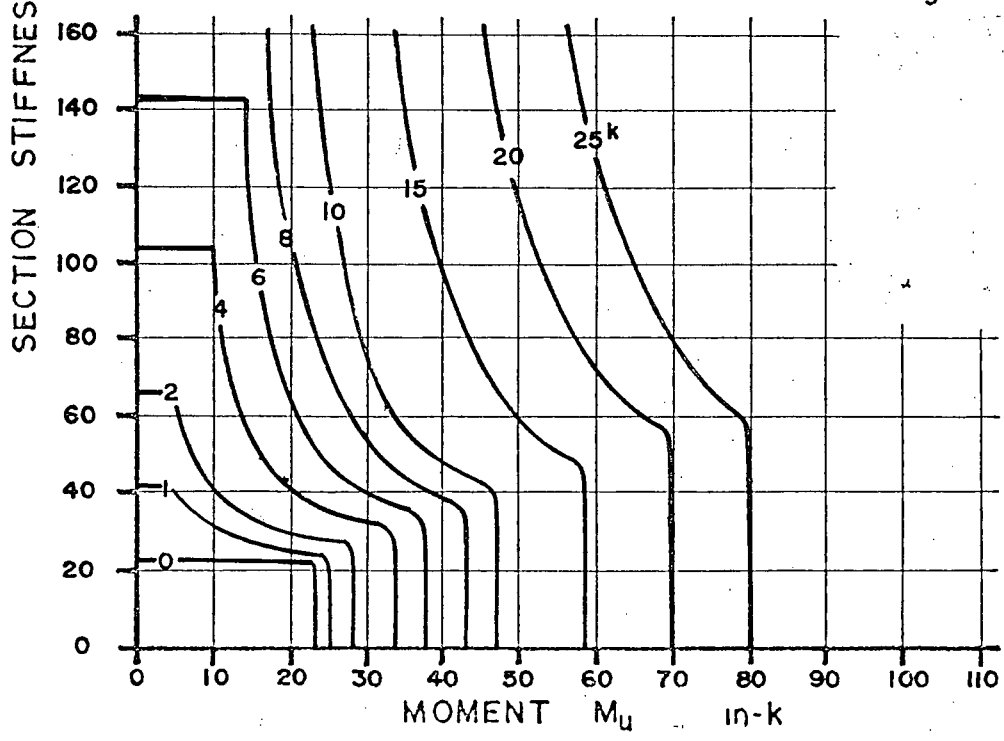


SECTION

$A_s = .10$ in²/ft



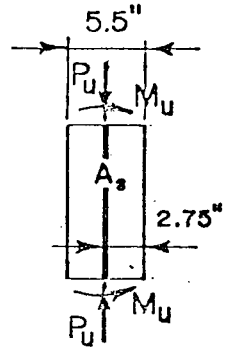
$A_s = .15$ in²/ft



CONCRETE WALL PANEL
SECTION STIFFNESS PROPERTIES

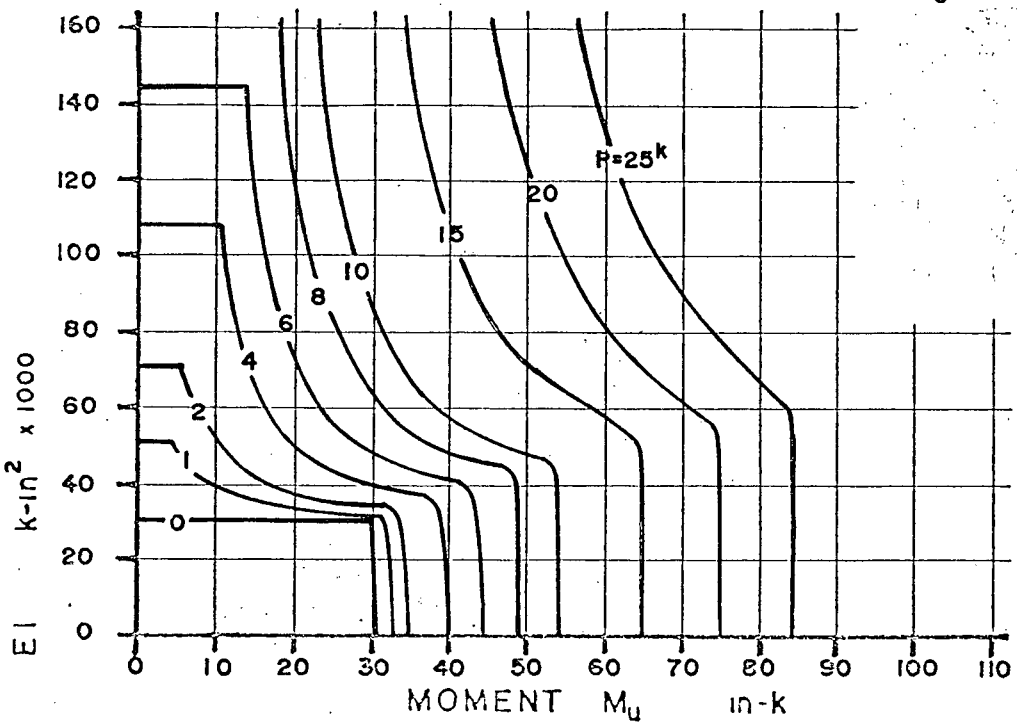
$f'_c = 4000$ PSI

$f_y = 60000$ PSI

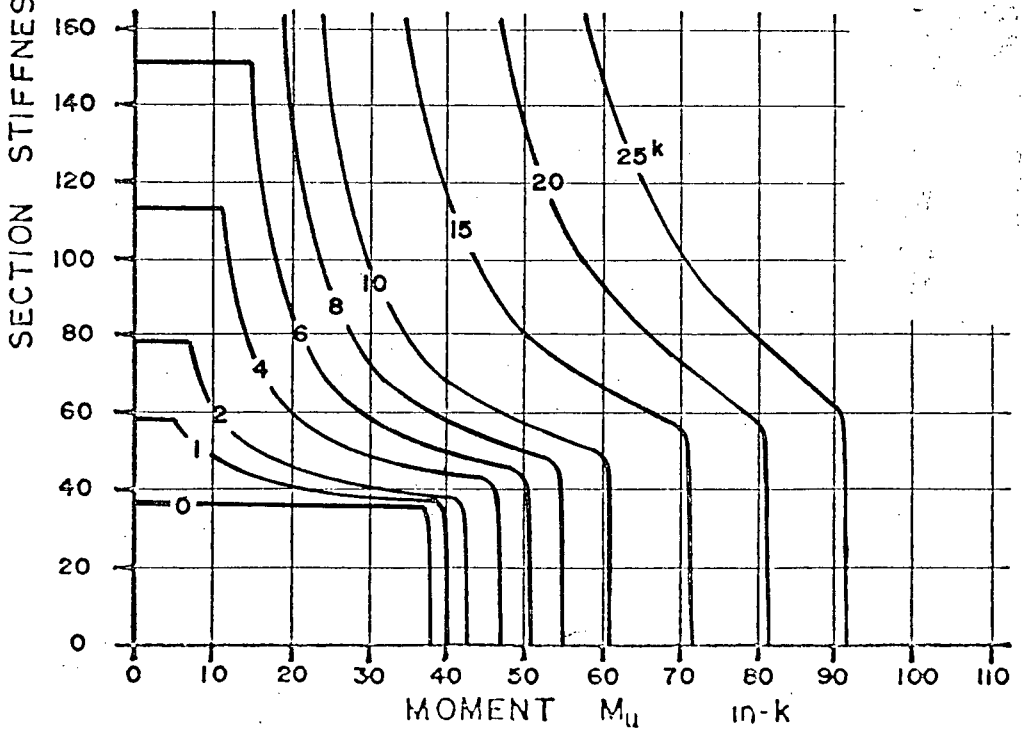


SECTION

$A_s = .20$ in²/ft



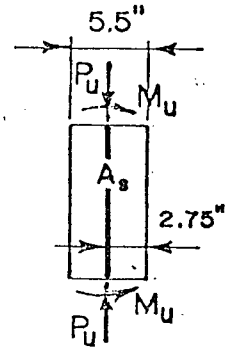
$A_s = .25$ in²/ft



CONCRETE WALL PANEL
SECTION STIFFNESS PROPERTIES

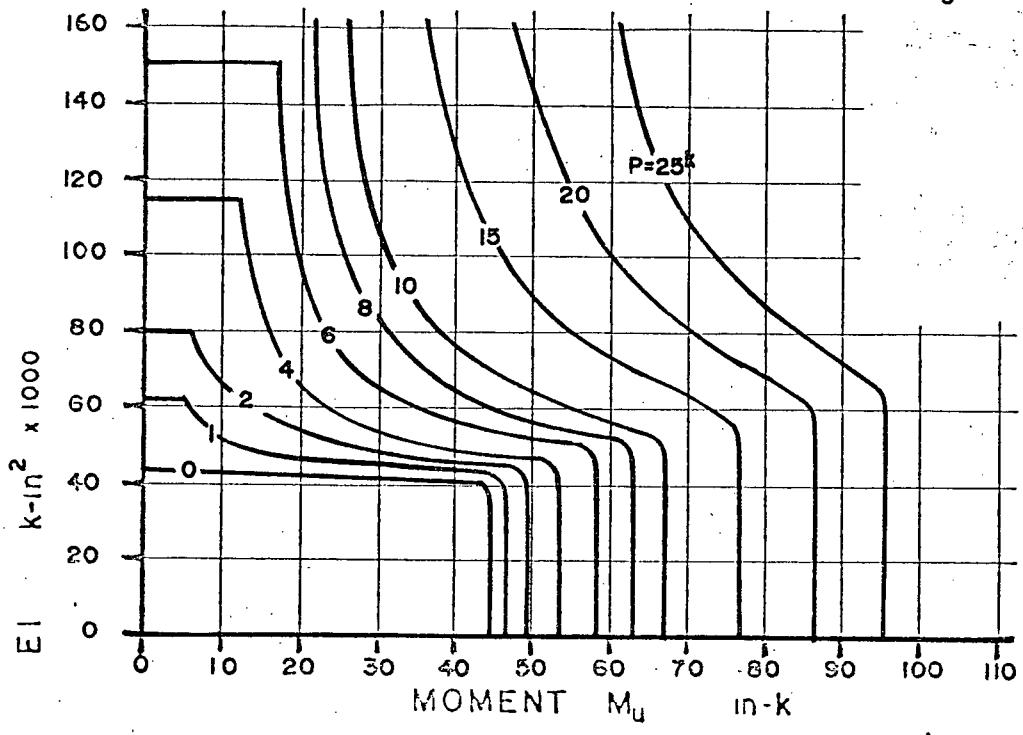
$f'_c = 4000$ PSI

$f_y = 60000$ PSI

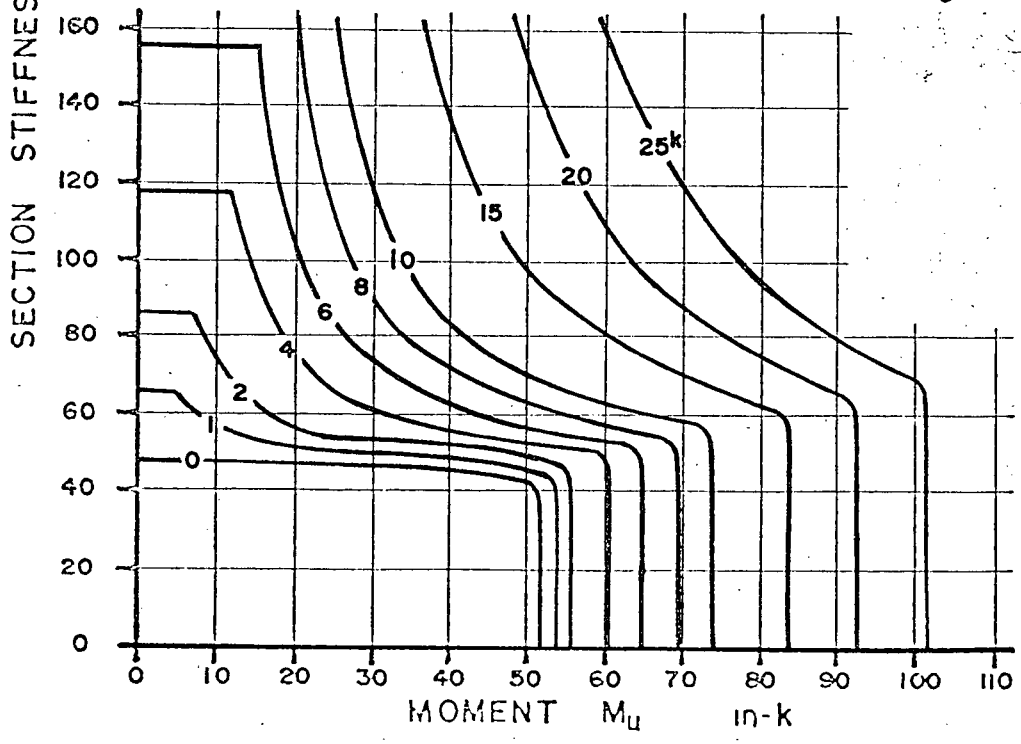


SECTION

$A_s = .30$ in²/ft



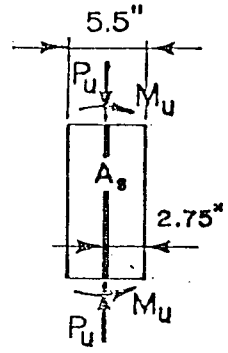
$A_s = .35$ in²/ft



CONCRETE WALL PANEL
SECTION STIFFNESS PROPERTIES

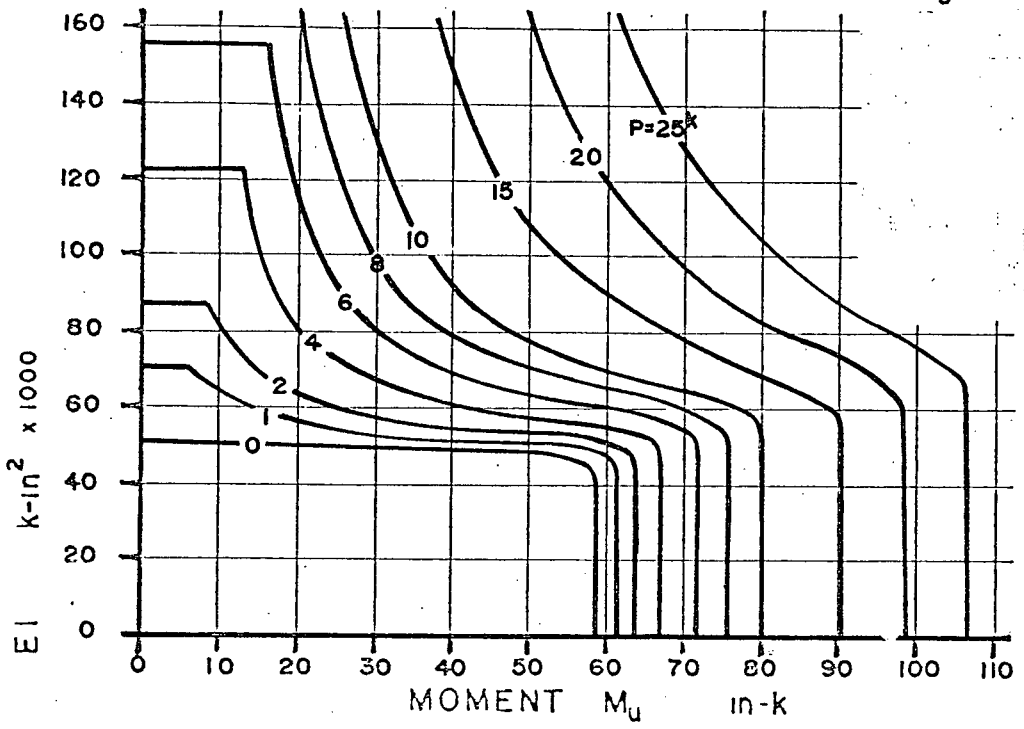
$f'_c = 4000$ PSI

$f_y = 60000$ PSI

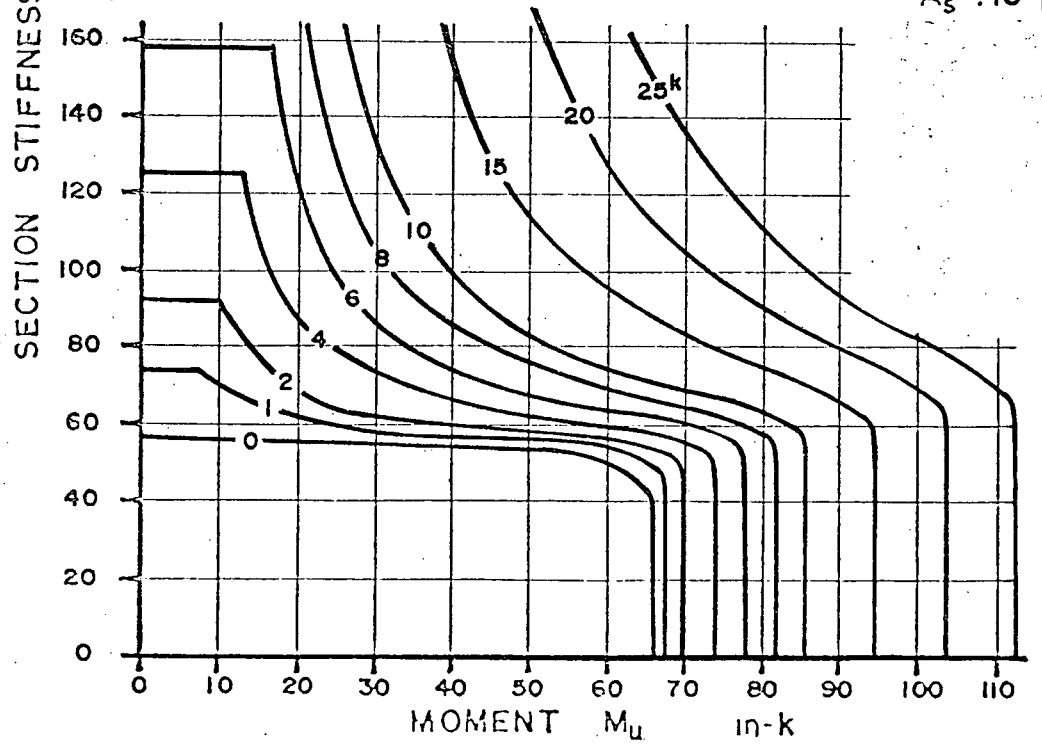


SECTION

$A_s = .40$ in²/ft



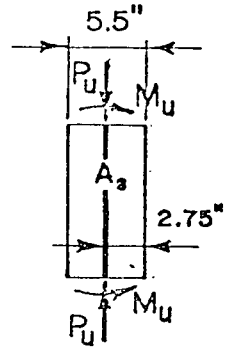
$A_s = .45$ in²/ft



CONCRETE WALL PANEL
SECTION STIFFNESS PROPERTIES

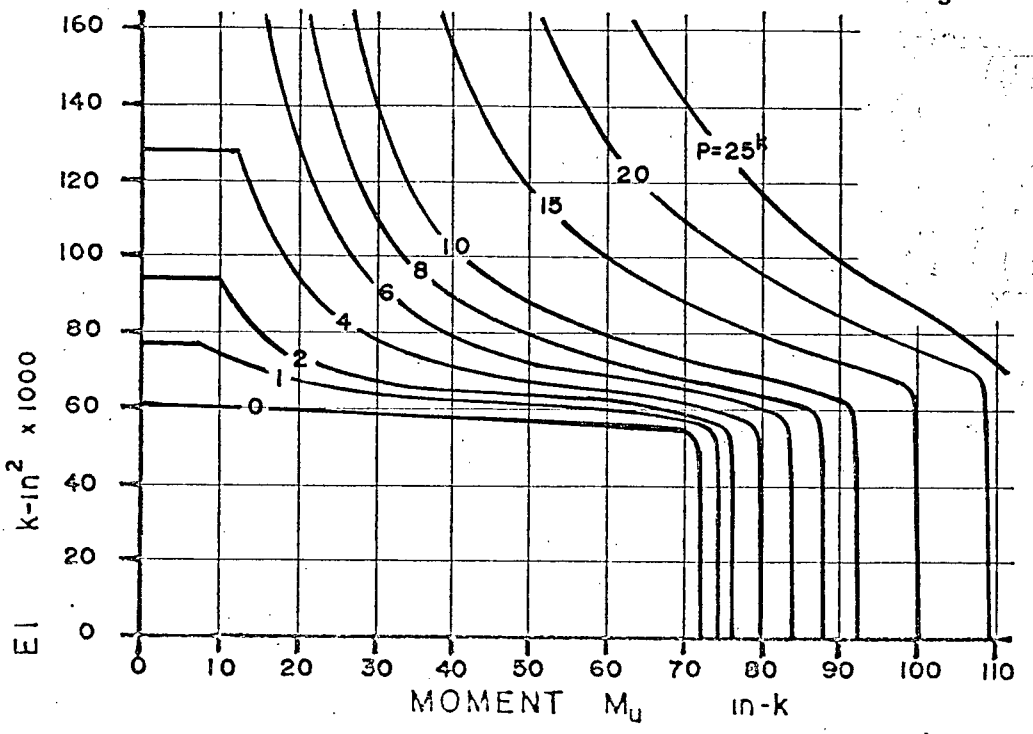
$f'_c = 4000$ PSI

$f_y = 60000$ PSI

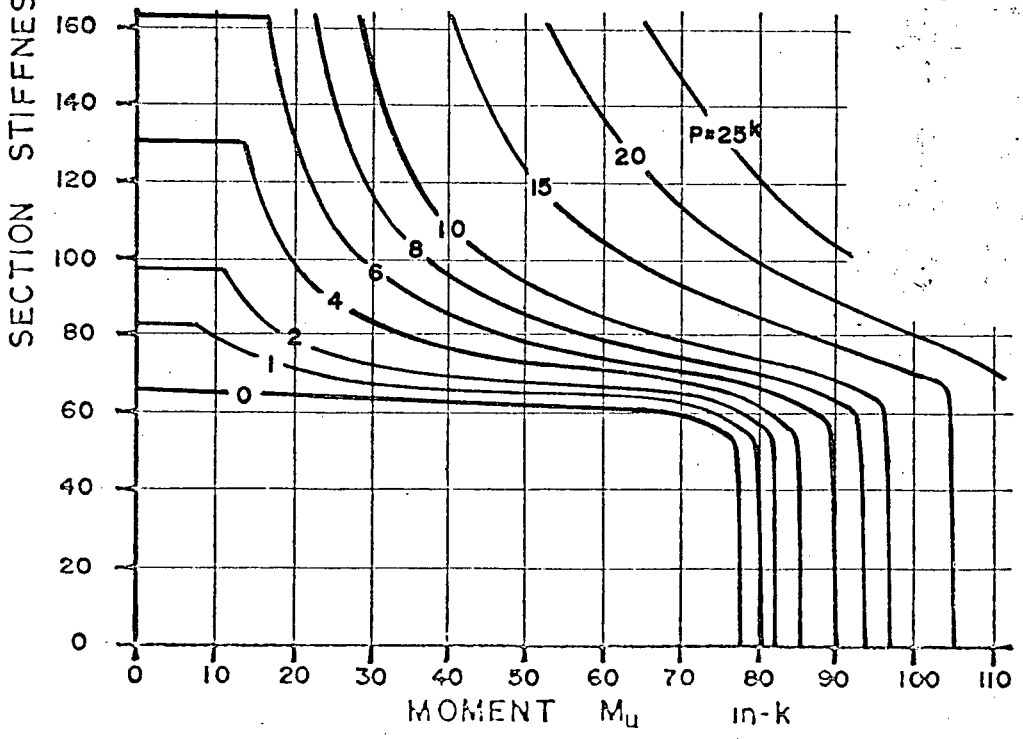


SECTION

$A_s = .50$ in^2/ft



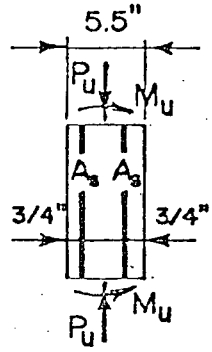
$A_s = .55$ in^2/ft



CONCRETE WALL PANEL
SECTION STIFFNESS PROPERTIES

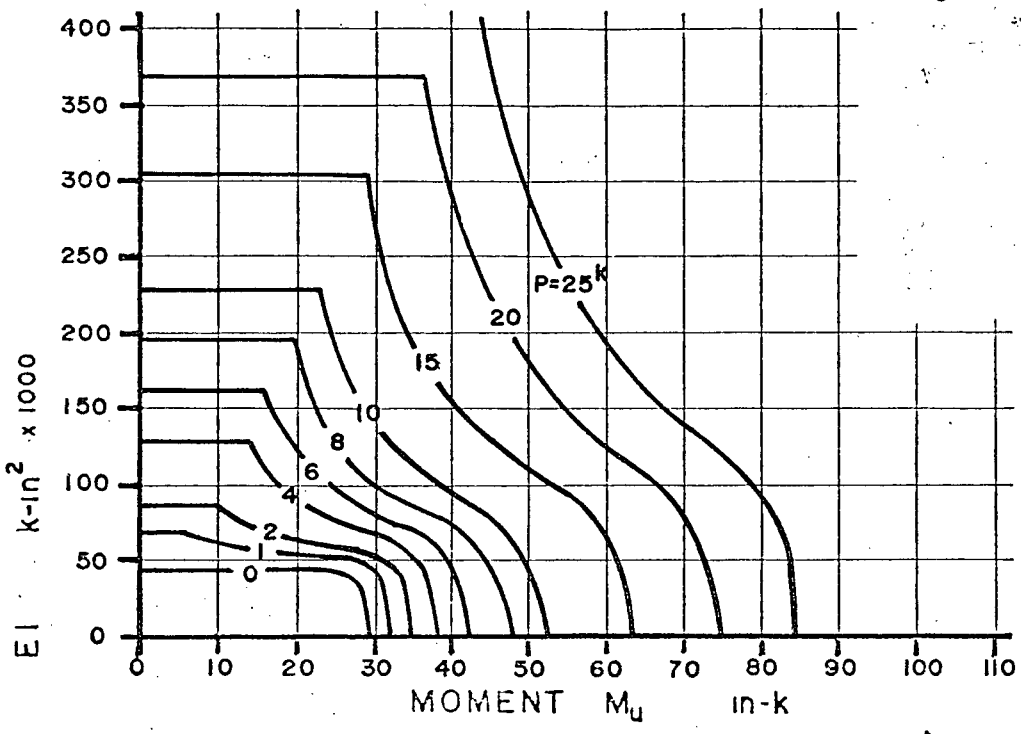
$f'_c = 4000$ PSI

$f_y = 60000$ PSI

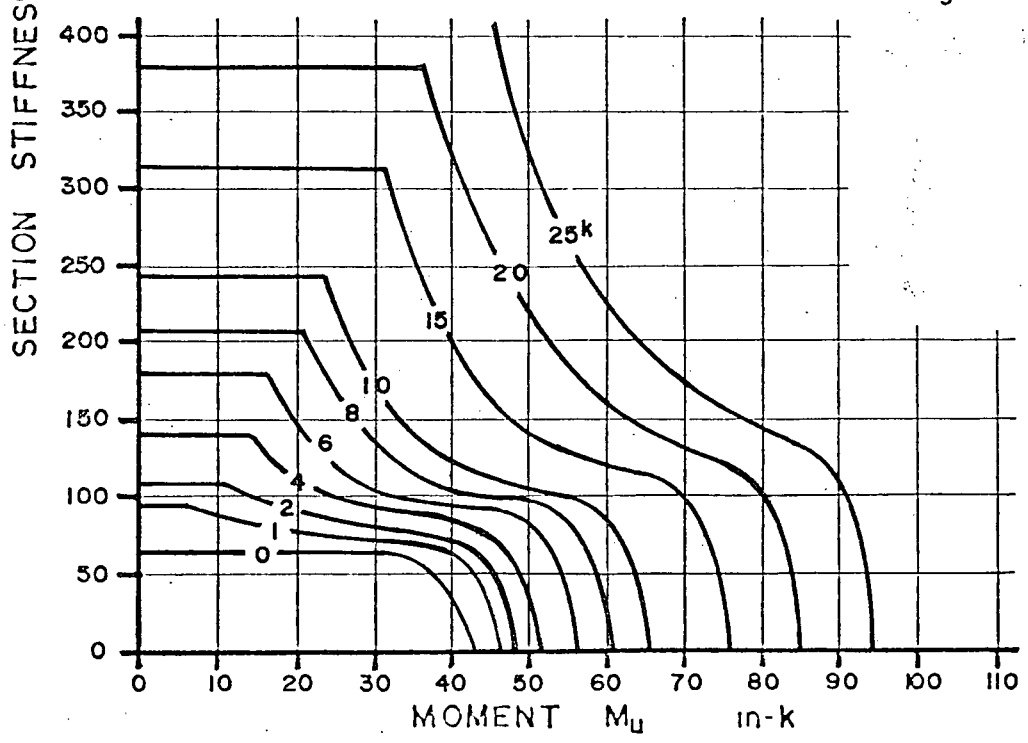


SECTION

$A_s = .10$ in²/ft



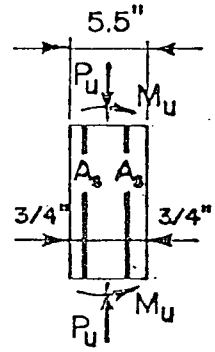
$A_s = .15$ in²/ft



CONCRETE WALL PANEL
SECTION STIFFNESS PROPERTIES

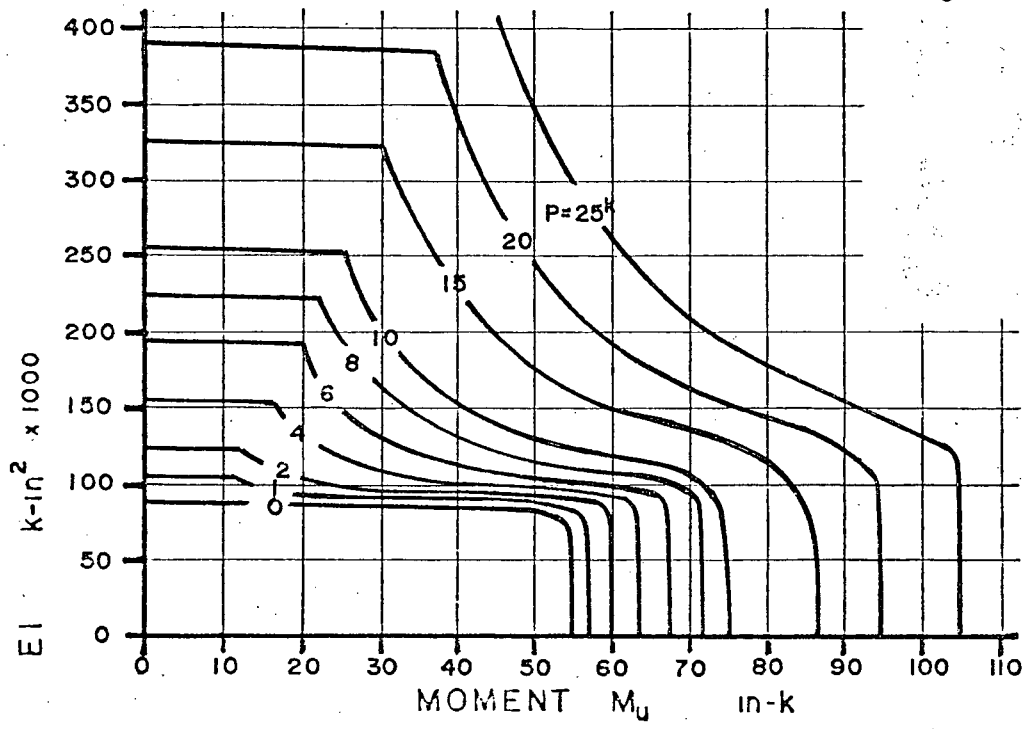
$f'_c = 4000$ PSI

$f_y = 60000$ PSI

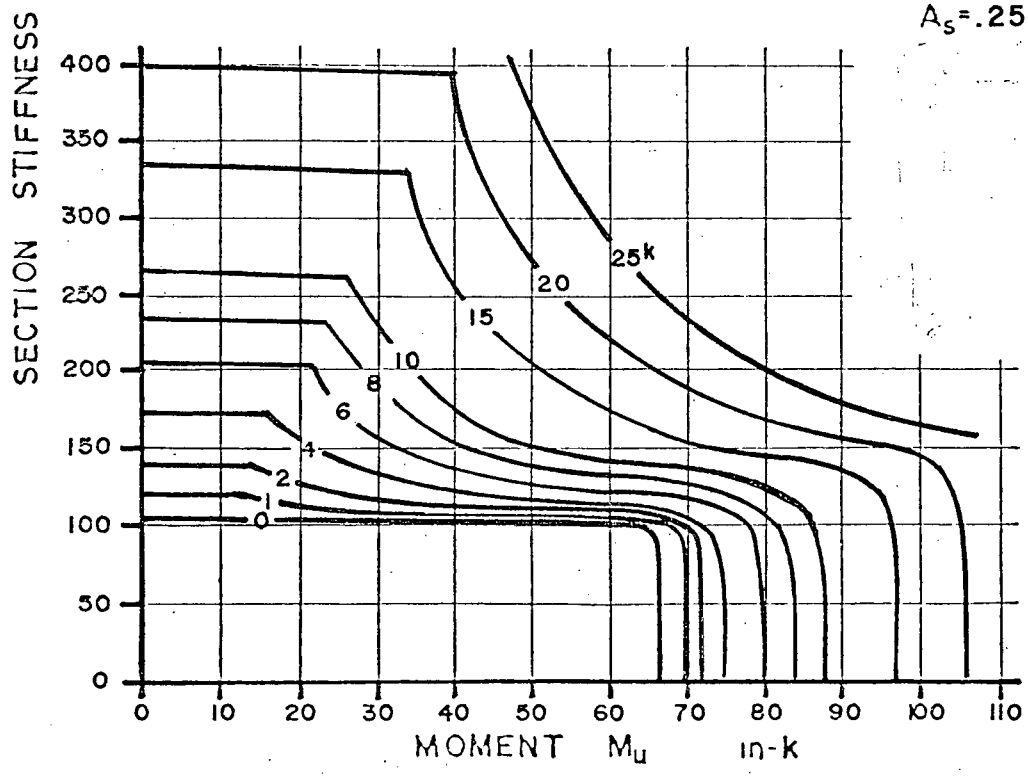


SECTION

$A_s = .20$ in^2/ft



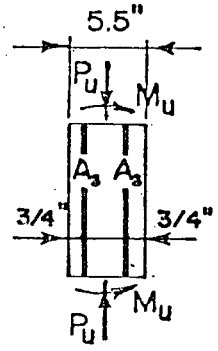
$A_s = .25$ in^2/ft



CONCRETE WALL PANEL
SECTION STIFFNESS PROPERTIES

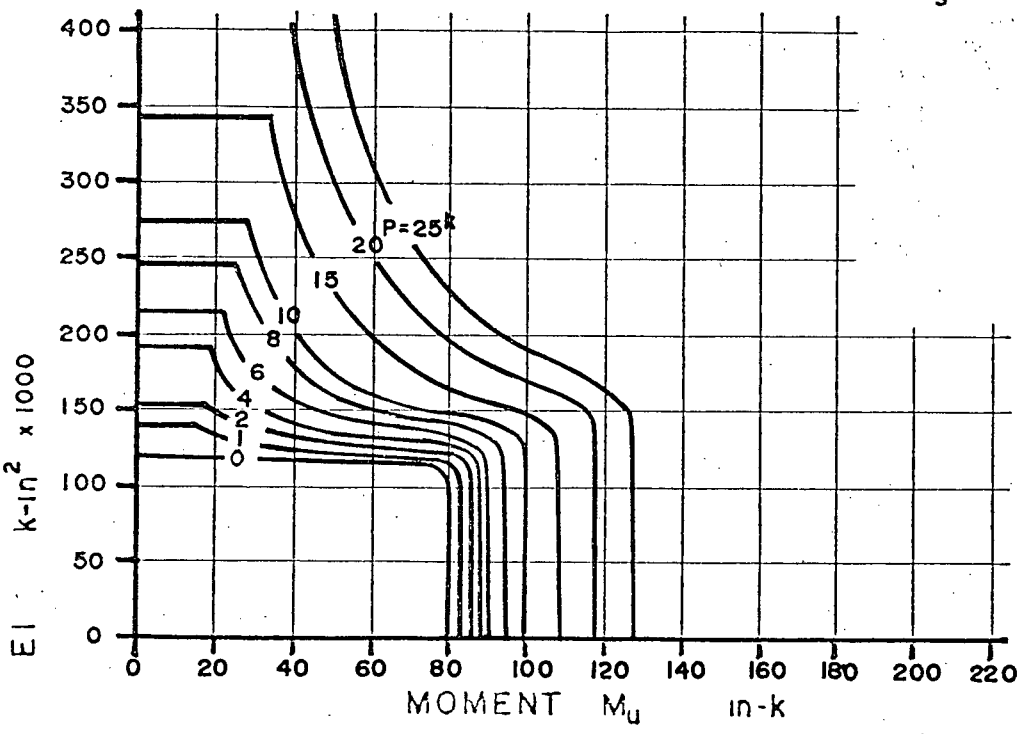
$f'_c = 4000$ PSI

$f_y = 60000$ PSI

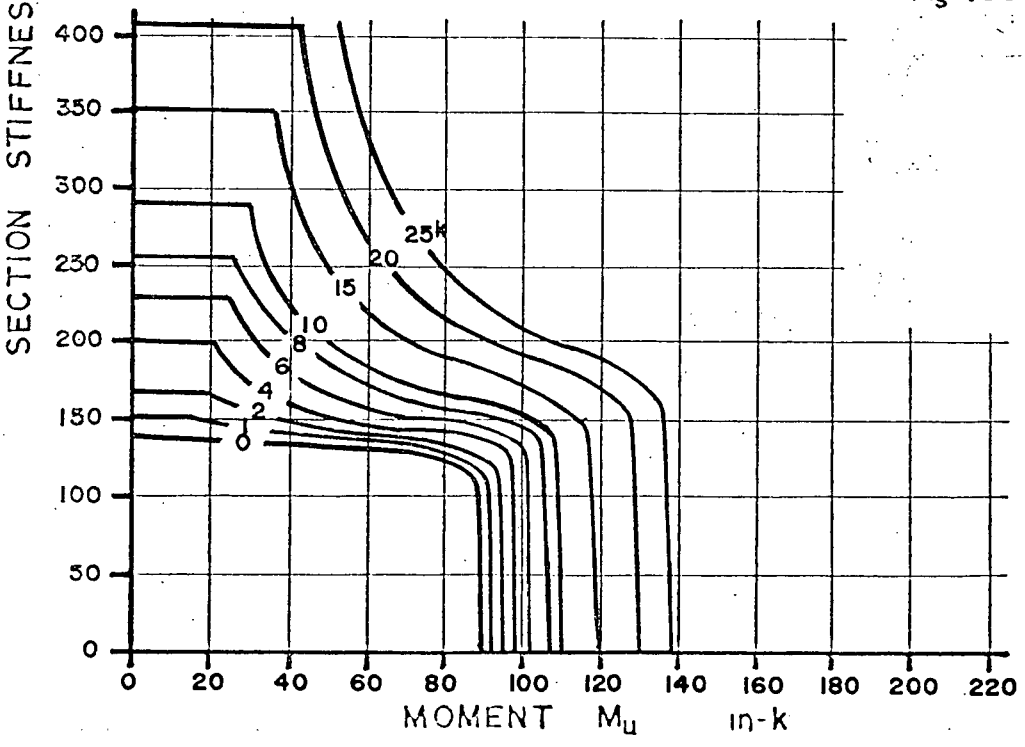


SECTION

$A_s = .30$ in²/ft



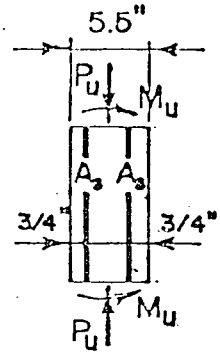
$A_s = .35$ in²/ft



CONCRETE WALL PANEL
SECTION STIFFNESS PROPERTIES

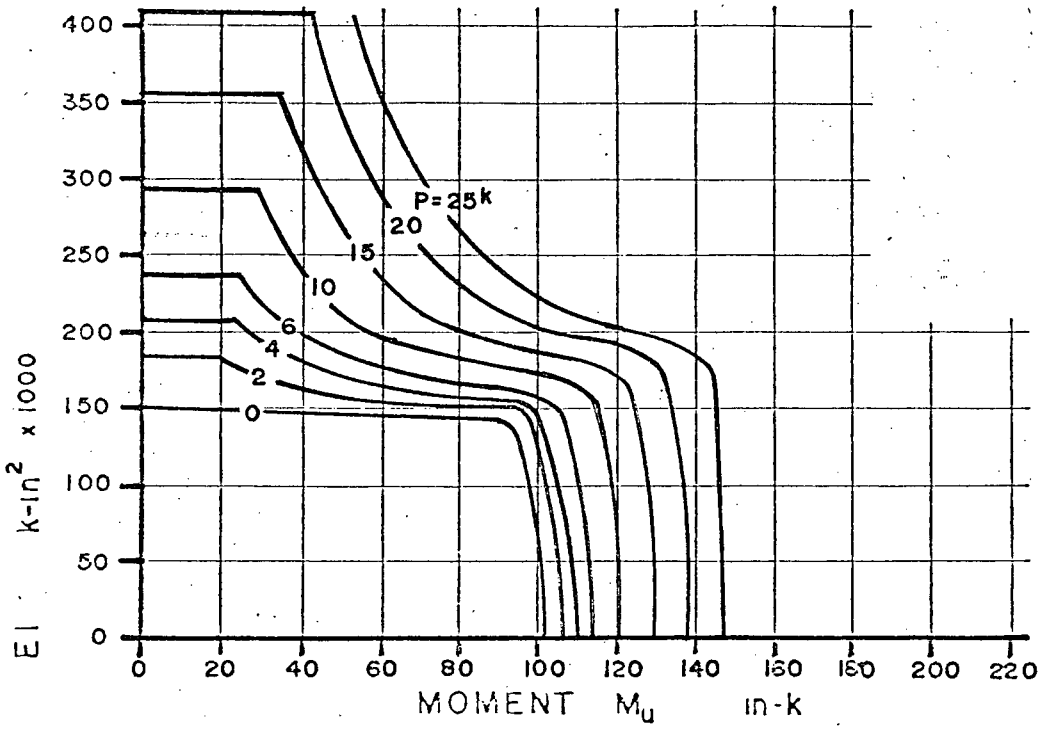
$f'_c = 4000$ PSI

$f_y = 60000$ PSI

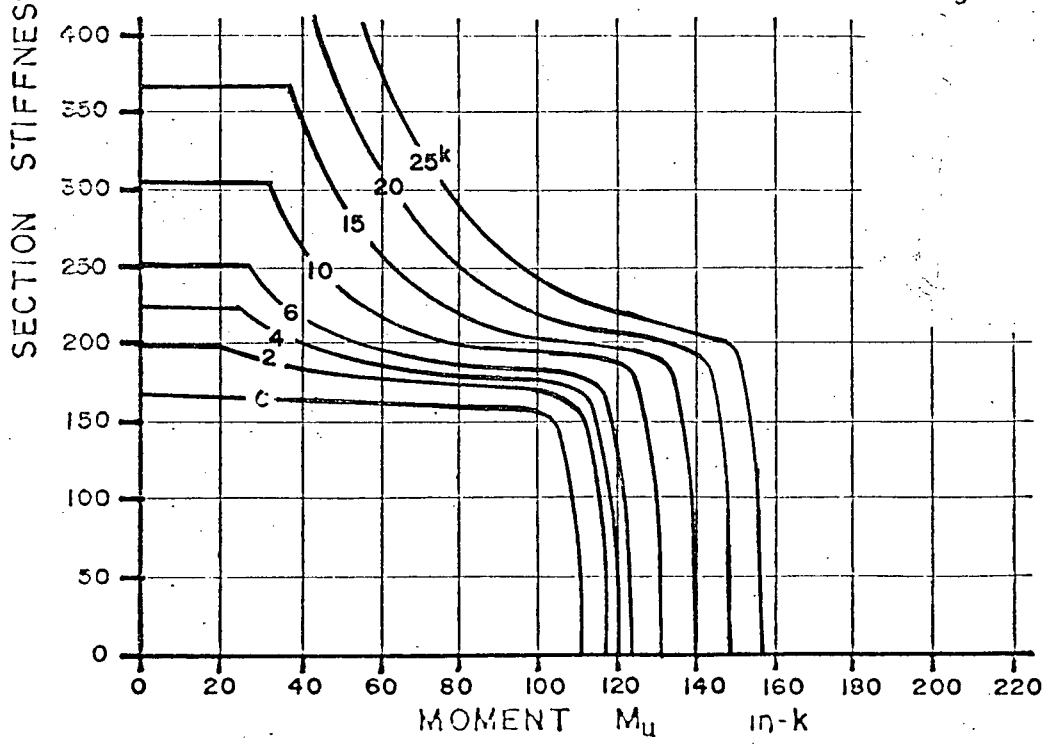


SECTION

$A_s = .40$ in^2/ft



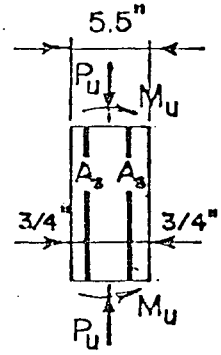
$A_s = .45$ in^2/ft



CONCRETE WALL PANEL
SECTION STIFFNESS PROPERTIES

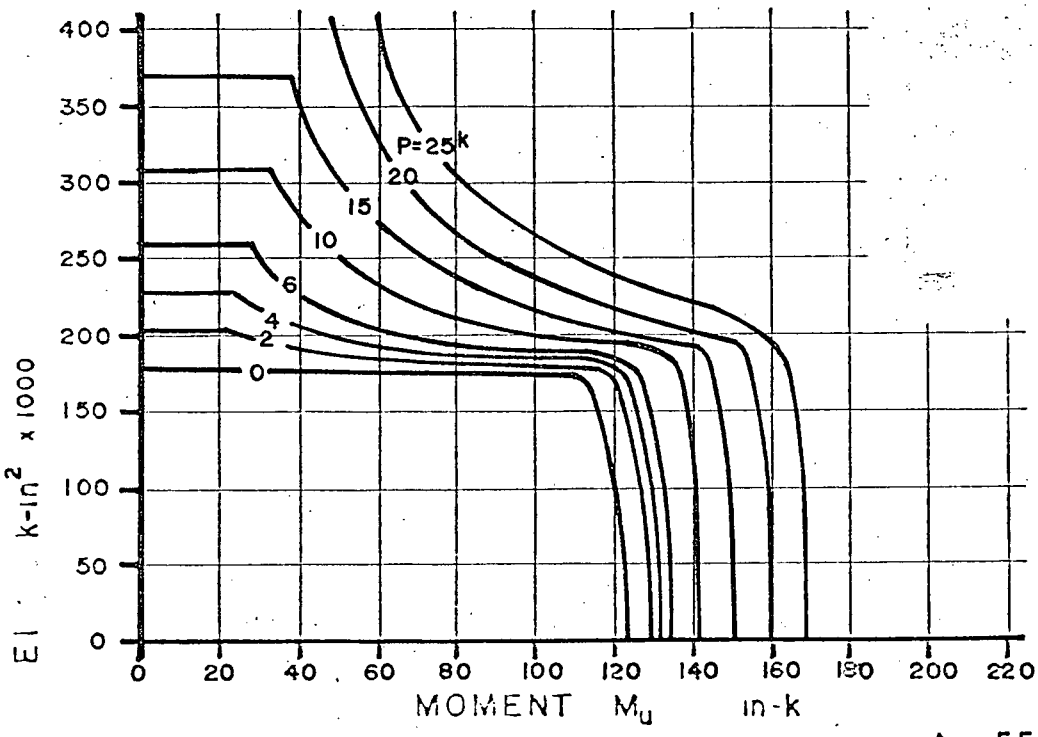
$f'_c = 4000$ PSI

$f_y = 60000$ PSI

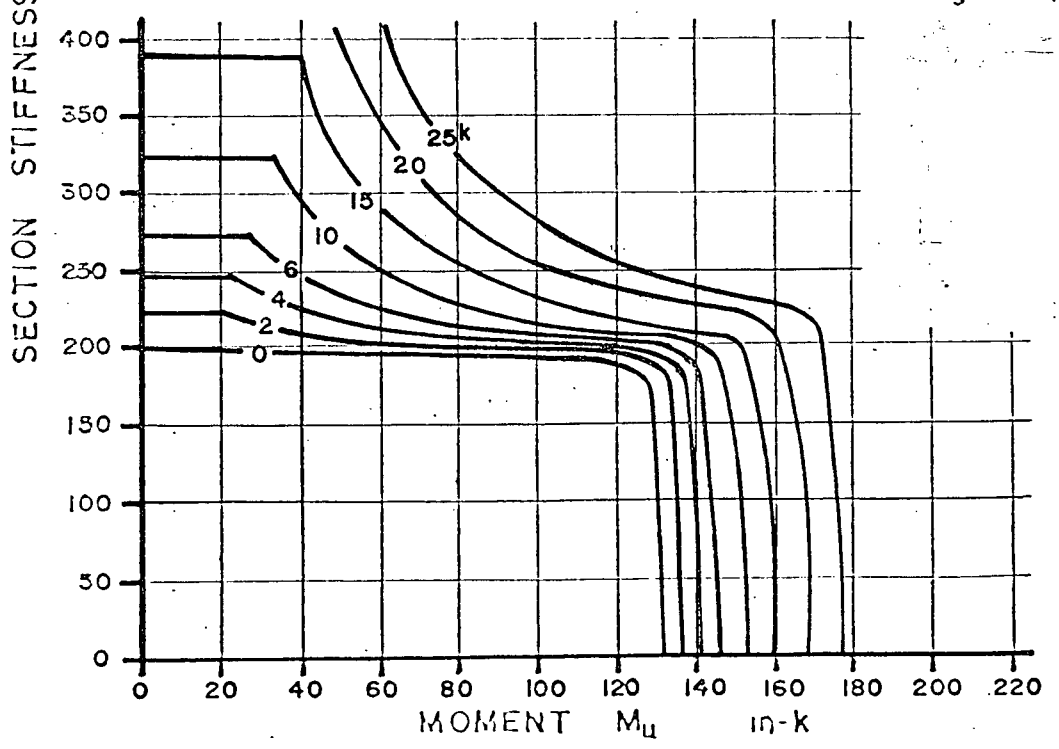


SECTION

$A_s = .50$ in²/ft



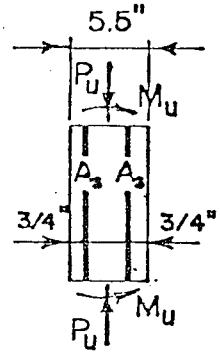
$A_s = .55$ in²/ft



CONCRETE WALL PANEL
SECTION STIFFNESS PROPERTIES

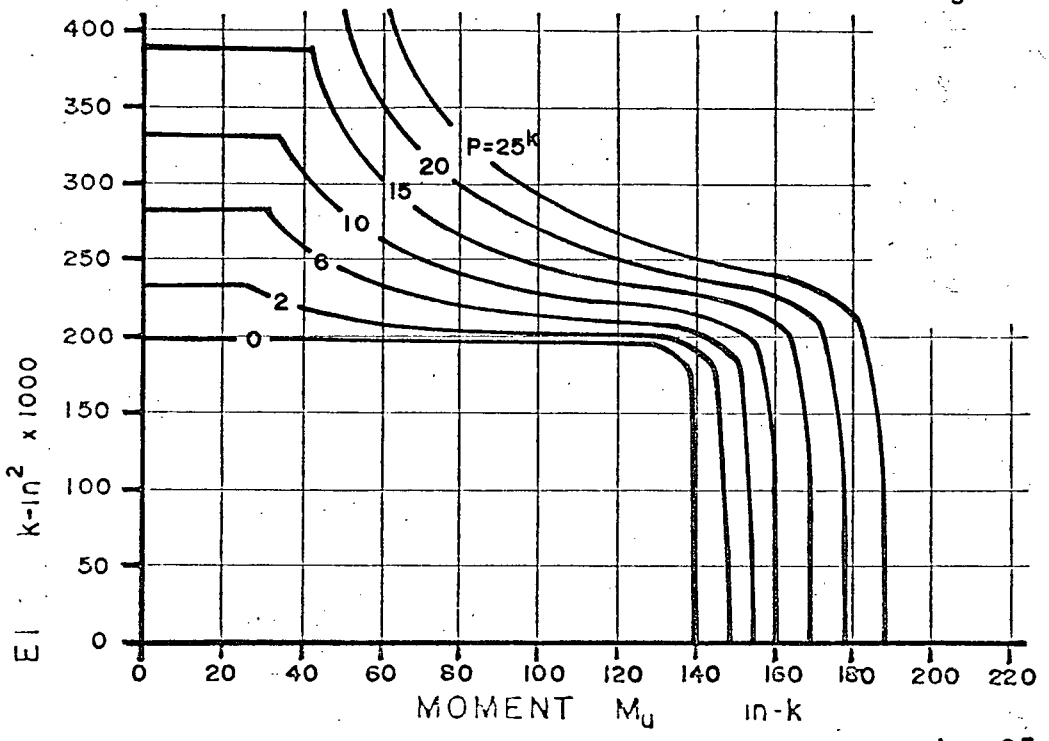
$f'_c = 4000$ PSI

$f_y = 60000$ PSI

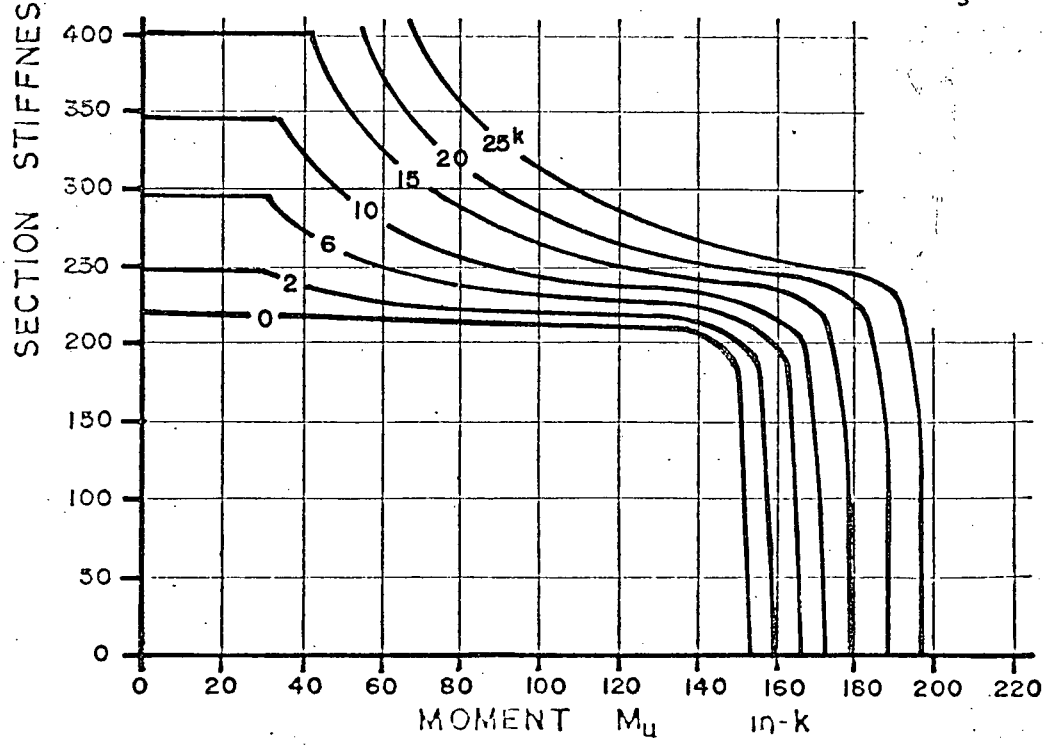


SECTION

$A_s = .60$ in^2/ft



$A_s = .65$ in^2/ft



APPENDIX D

LOAD CAPACITY CHARTS

AXIAL LOAD/LATERAL LOAD

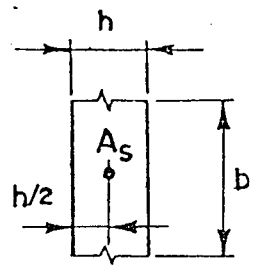
CONCRETE WALL PANEL AXIAL LOAD CAPACITY

$h = 5.5$ IN

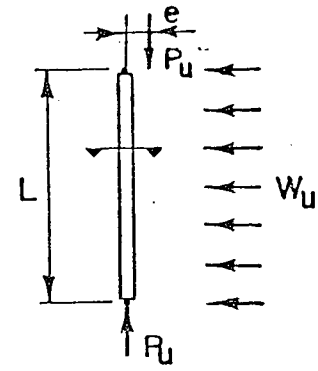
$A_s = .10$ IN^2/FT

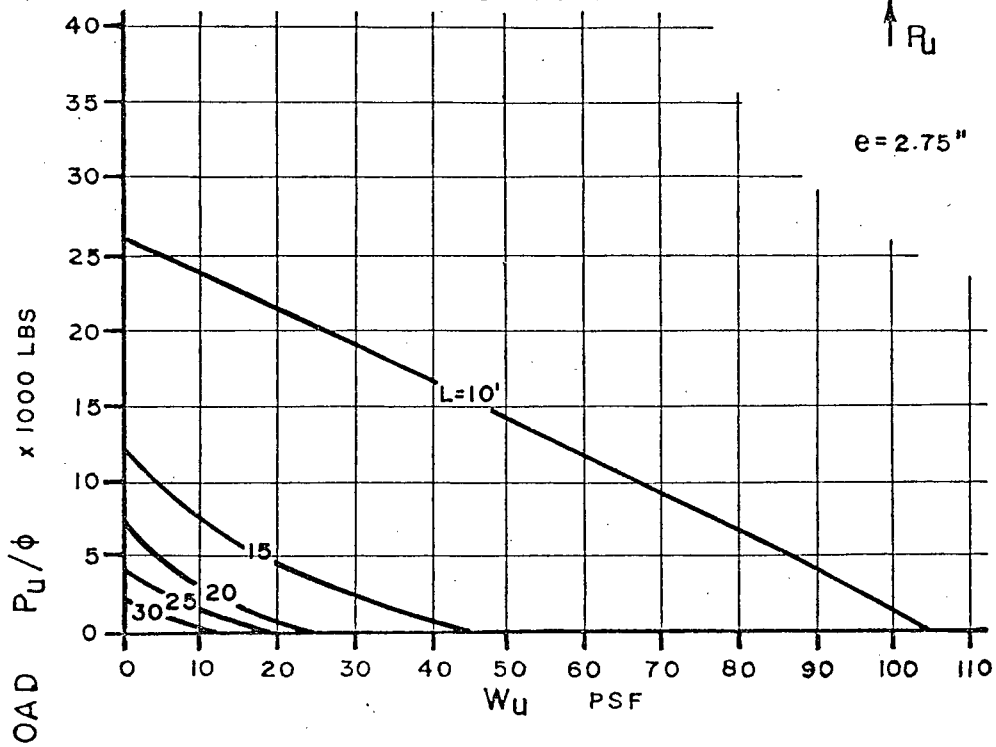
$f'_c = 4000$ PSI

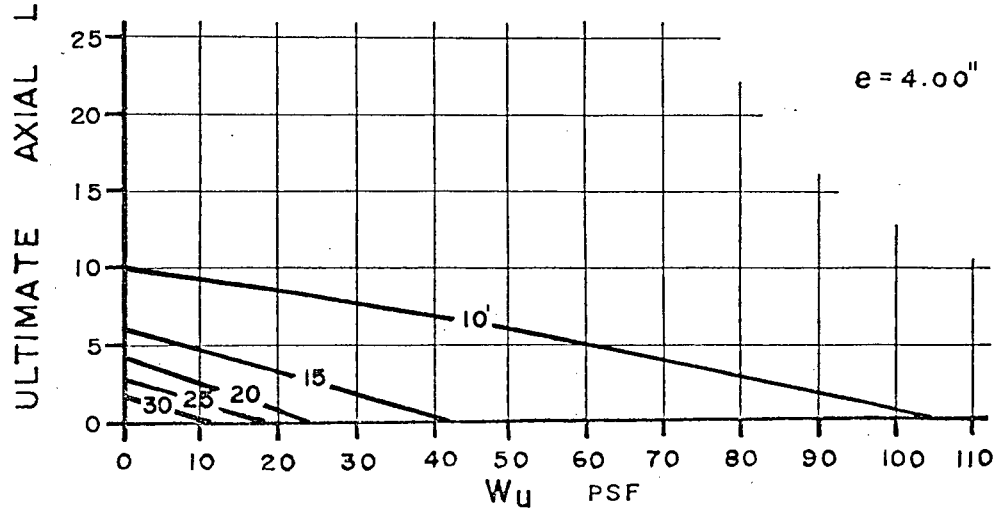
$f_y = 60000$ PSI

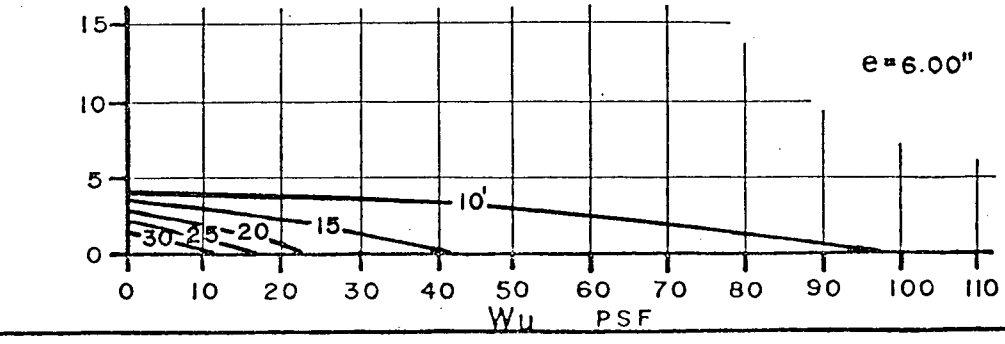


SECTION









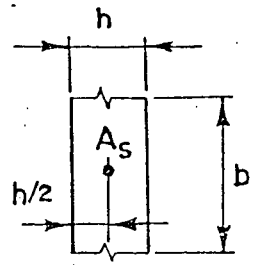
CONCRETE WALL PANEL AXIAL LOAD CAPACITY

$h = 5.5$ IN

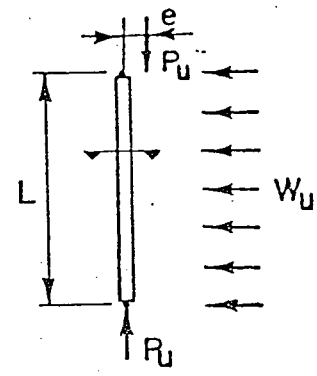
$A_s = .15$ IN^2/FT

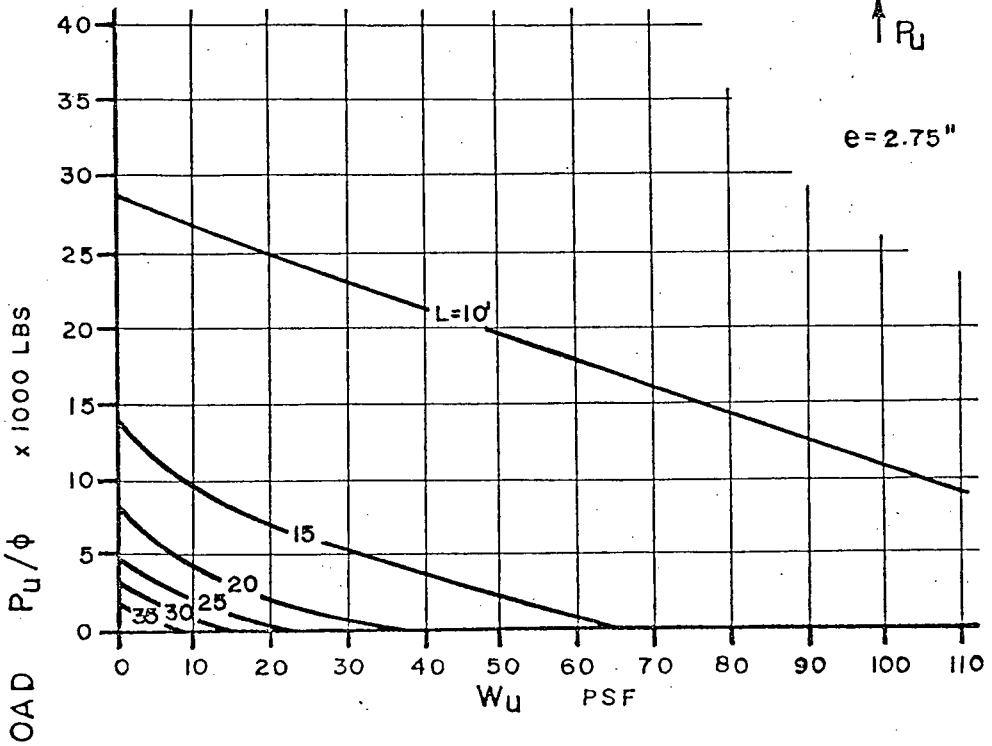
$f'_c = 4000$ PSI

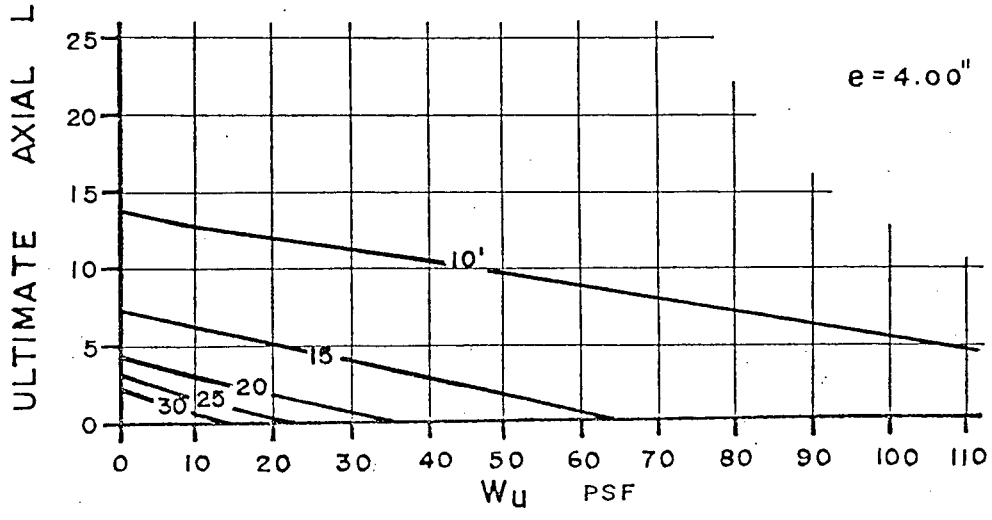
$f_y = 60000$ PSI

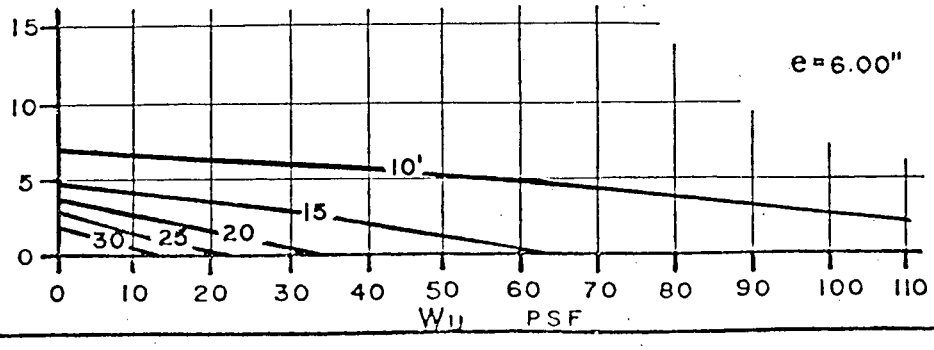


SECTION









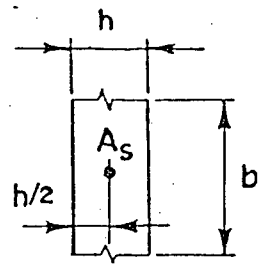
CONCRETE WALL PANEL
AXIAL LOAD CAPACITY

$h = 5.5$ IN

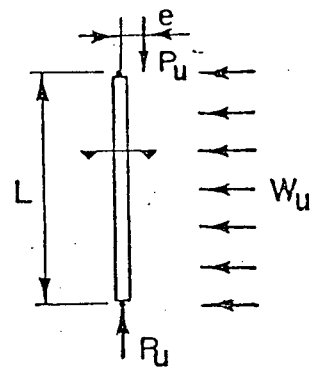
$A_s = .20$ IN^2/ FT

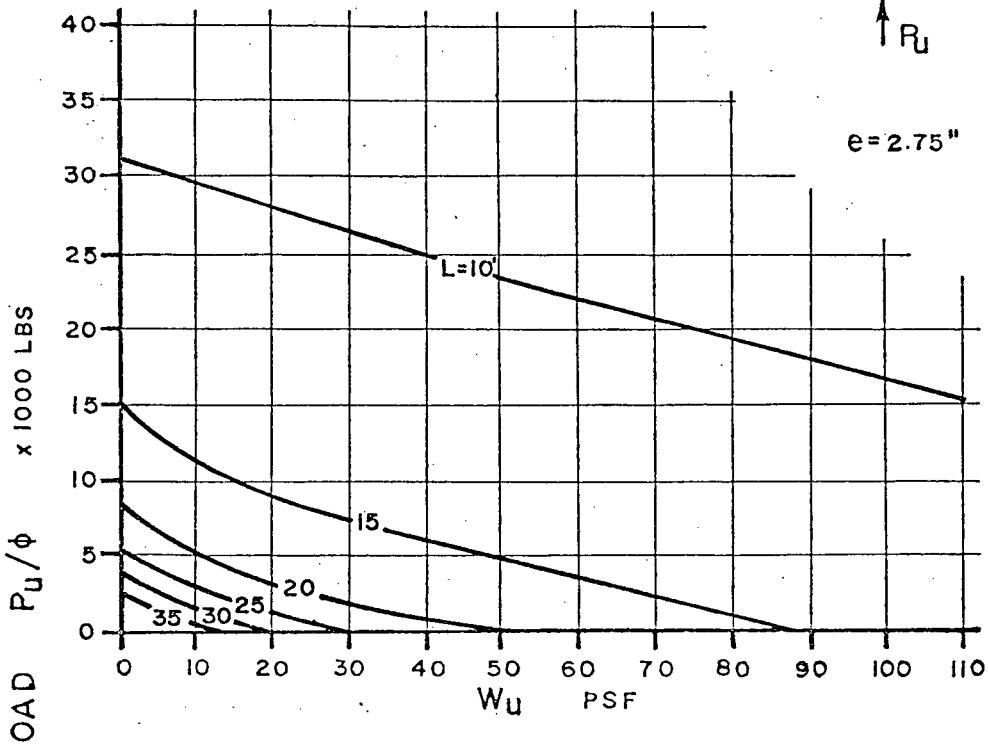
$f'_c = 4000$ PSI

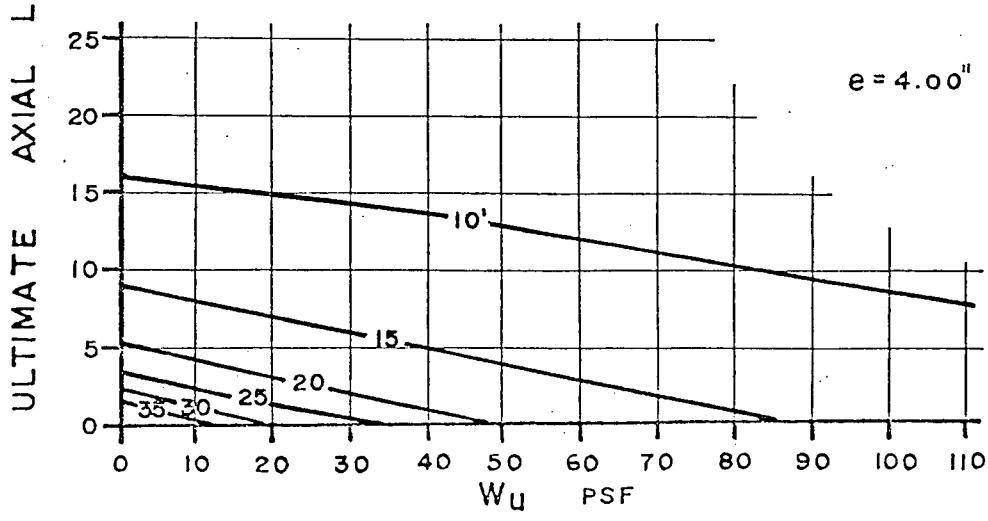
$f_y = 60000$ PSI

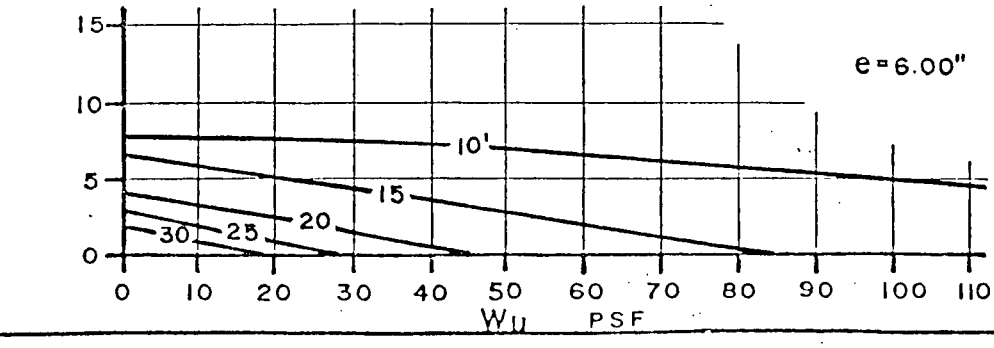


SECTION









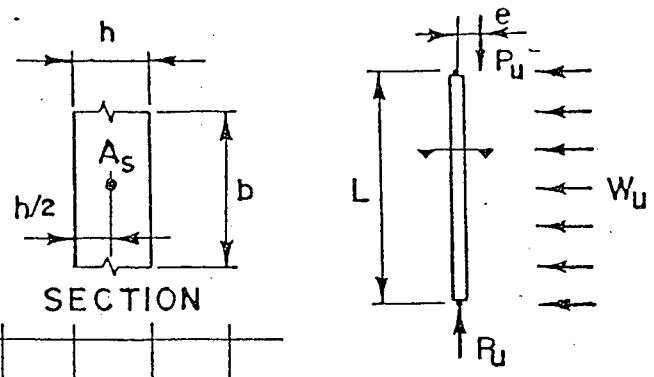
CONCRETE WALL PANEL AXIAL LOAD CAPACITY

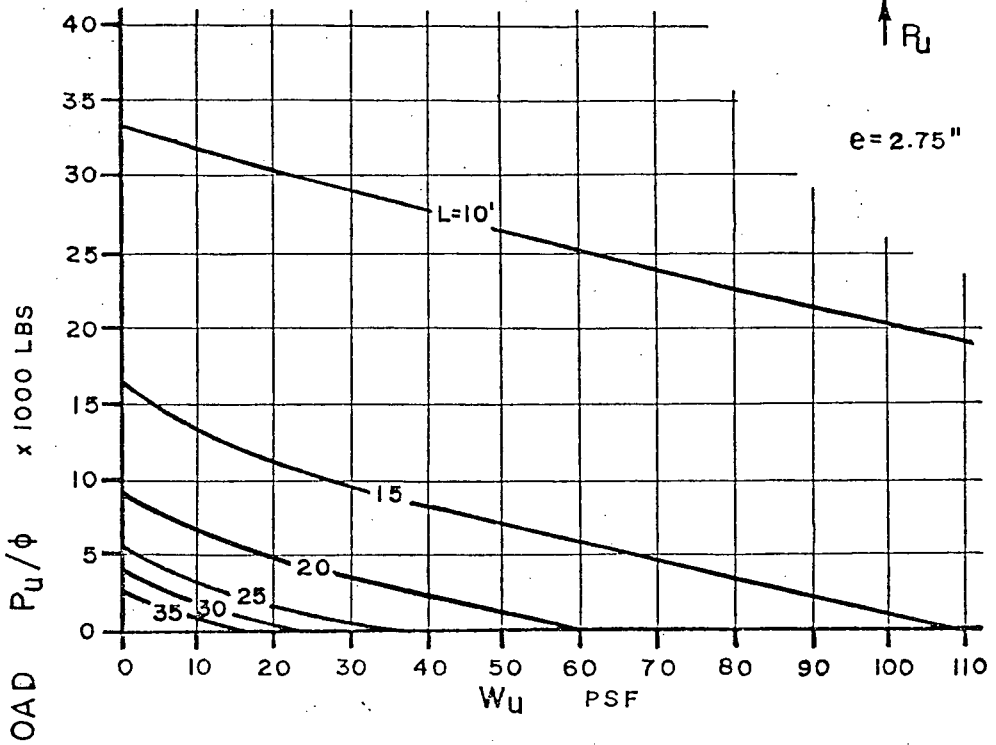
$h = 5.5$ IN

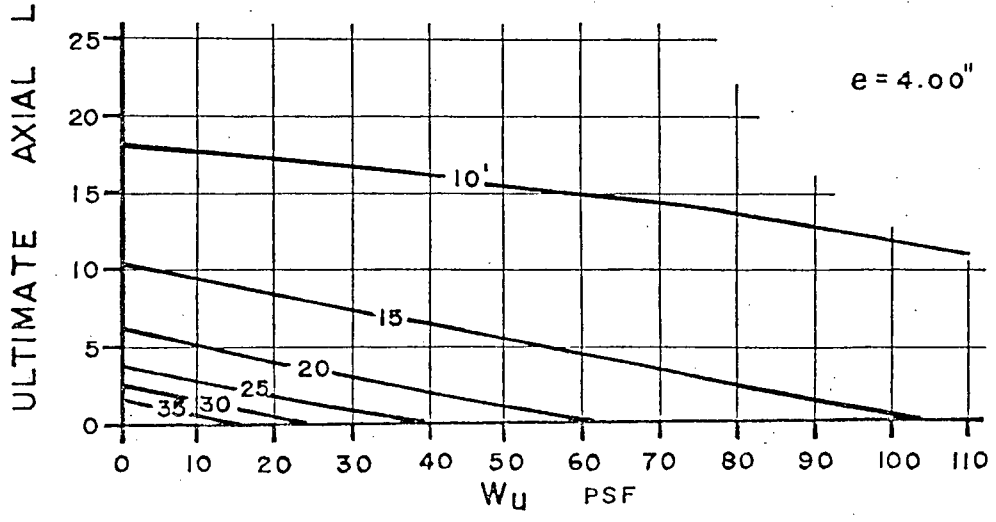
$A_s = .25$ IN^2/FT

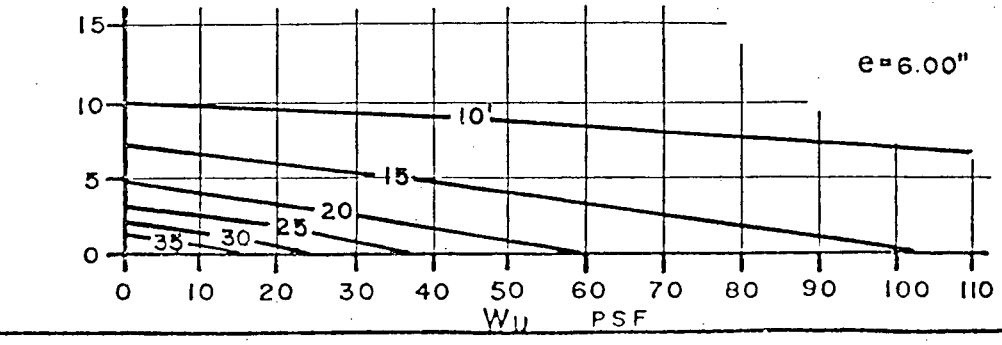
$f'_c = 4000$ PSI

$f_y = 60000$ PSI









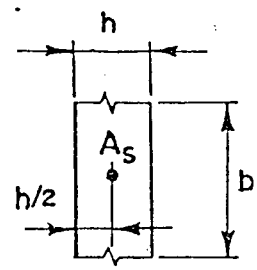
CONCRETE WALL PANEL
AXIAL LOAD CAPACITY

$h = 5.5$ IN

$A_s = .30$ IN^2/FT

$f'_c = 4000$ PSI

$f_y = 60000$ PSI



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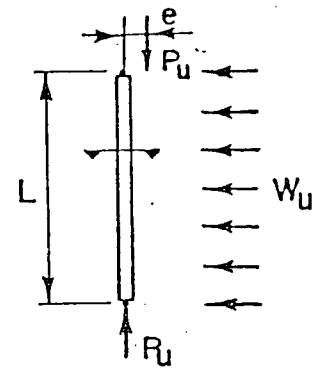
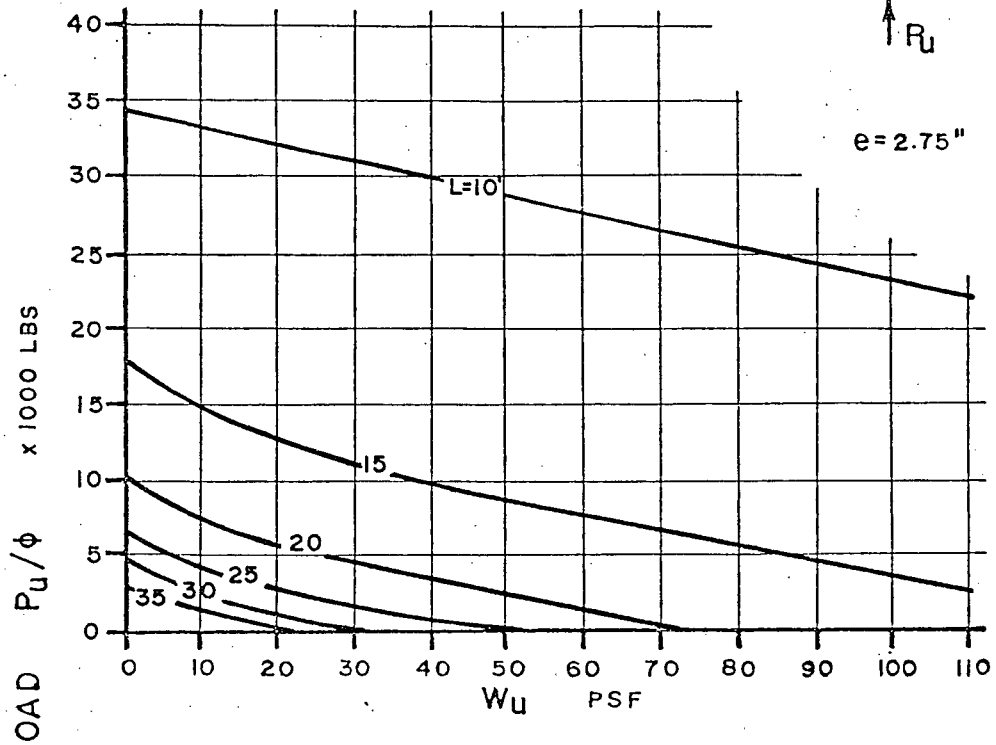
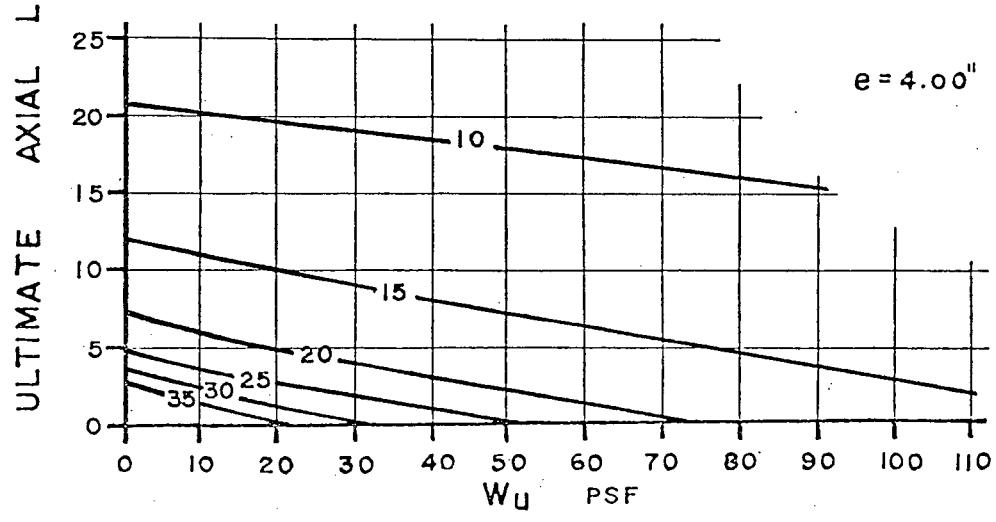
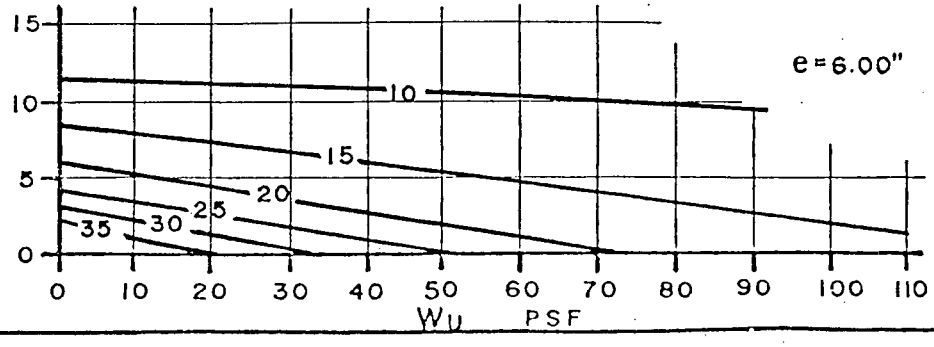








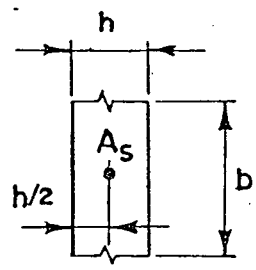
CONCRETE WALL PANEL: AXIAL LOAD CAPACITY

$h = 5.5$ IN

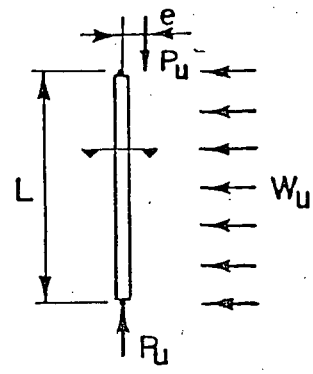
$A_s = .35$ IN^2/FT

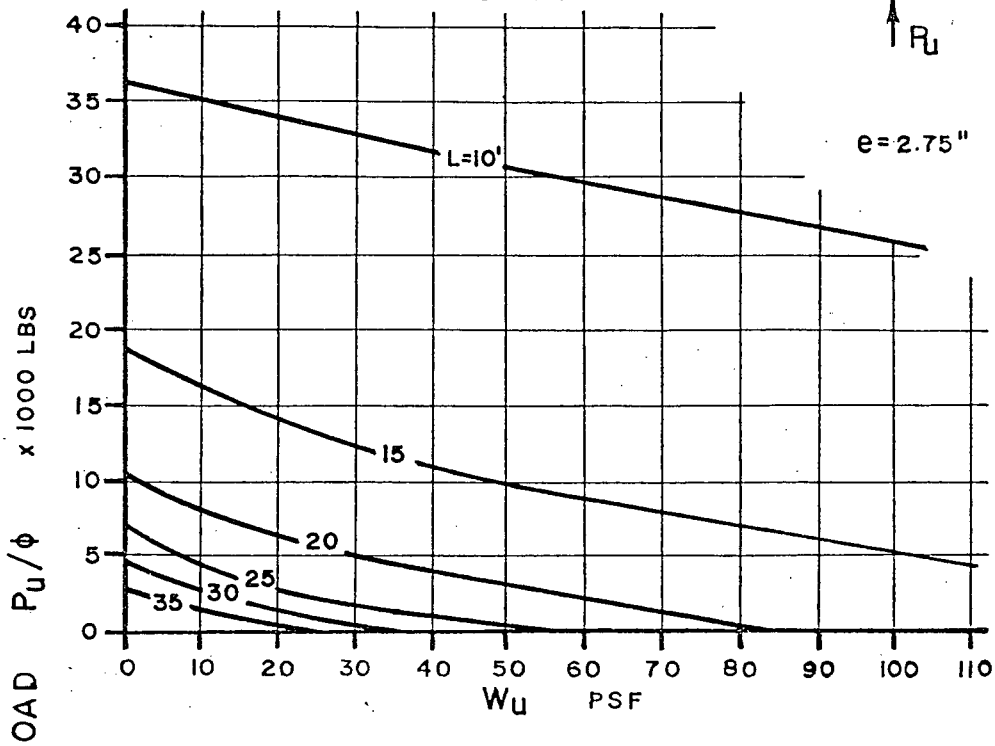
$f'_c = 4000$ PSI

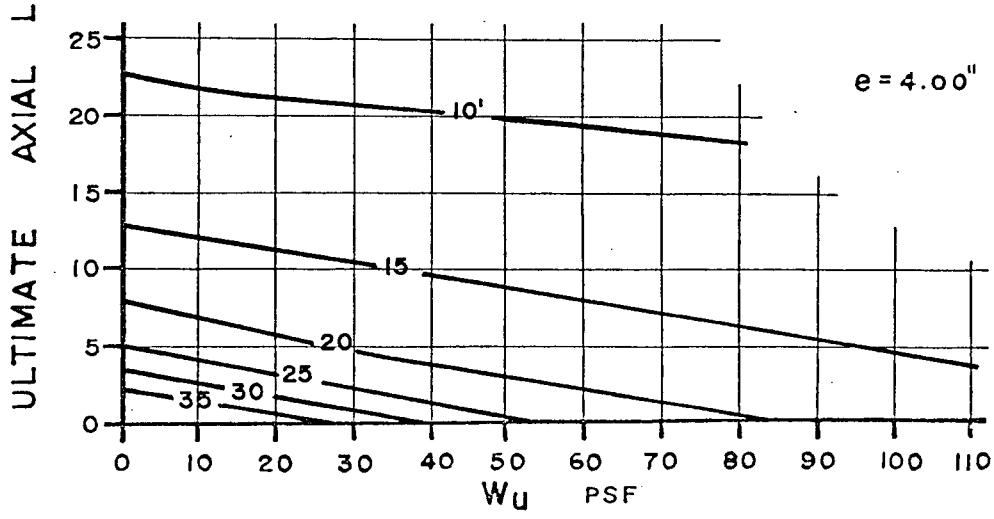
$f_y = 60000$ PSI

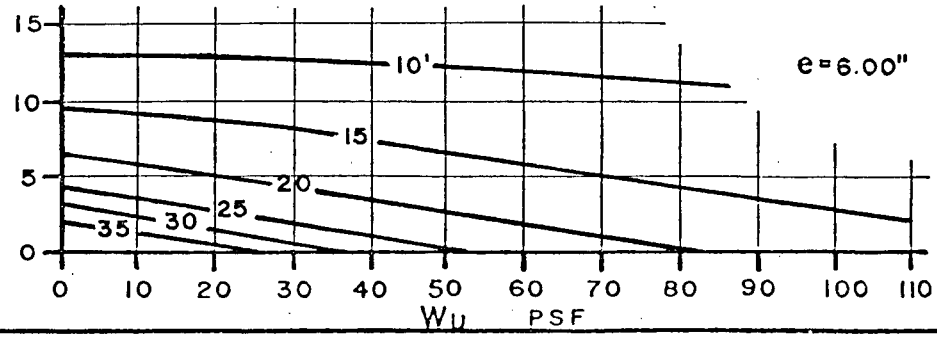


SECTION









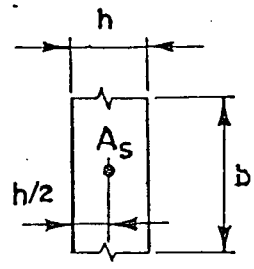
CONCRETE WALL PANEL
AXIAL LOAD CAPACITY

$h = 5.5$ IN

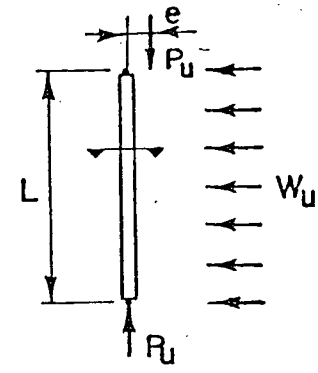
$A_s = .40$ IN^2/FT

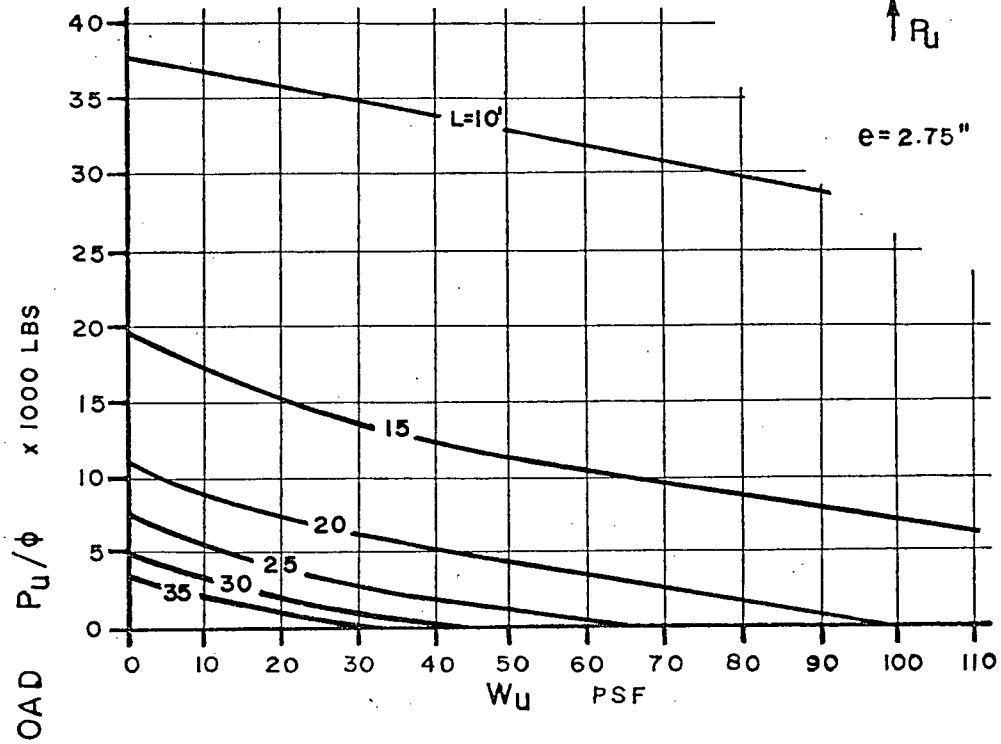
$f'_c = 4000$ PSI

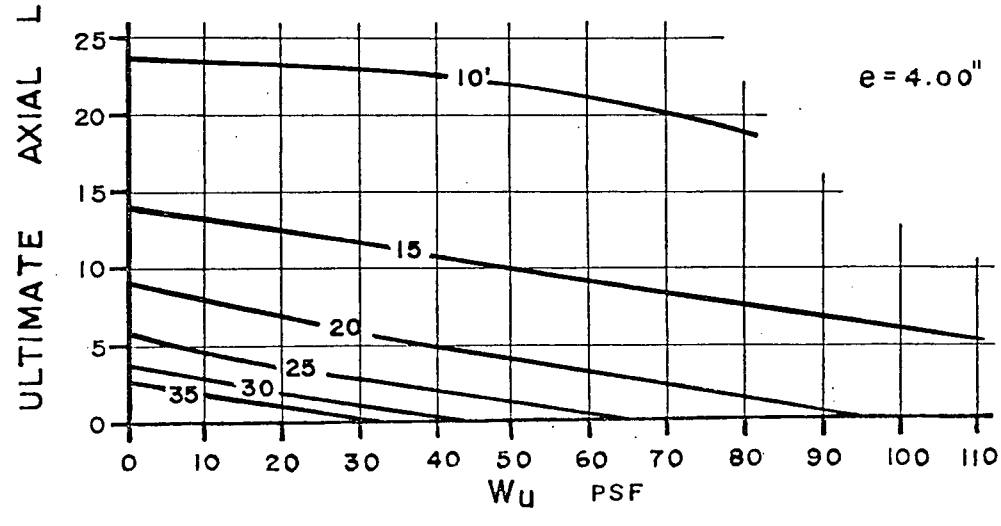
$f_y = 60000$ PSI

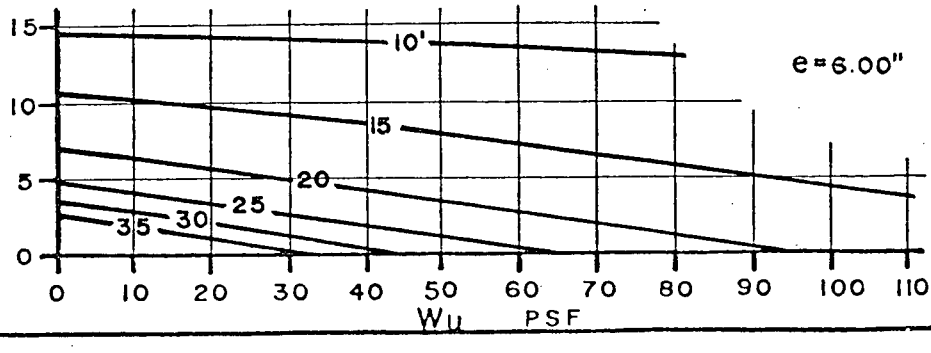


SECTION









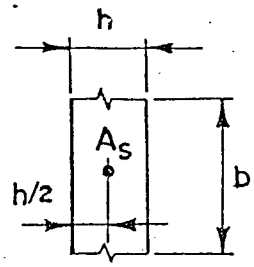
CONCRETE WALL PANEL
AXIAL LOAD CAPACITY

$h = 5.5$ IN

$A_s = .45$ IN^2/FT

$f'_c = 4000$ PSI

$f_y = 60000$ PSI



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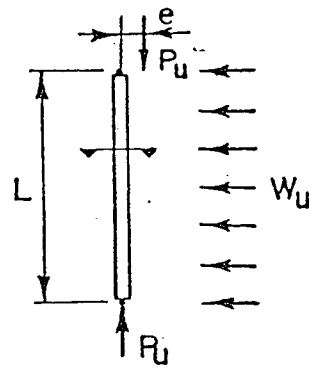
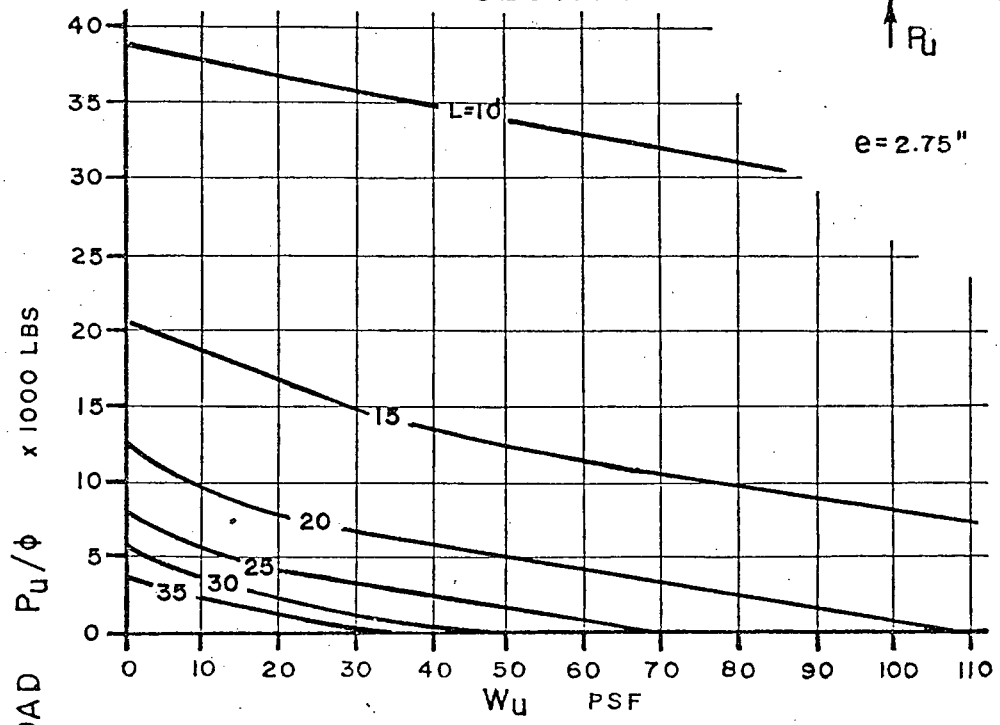
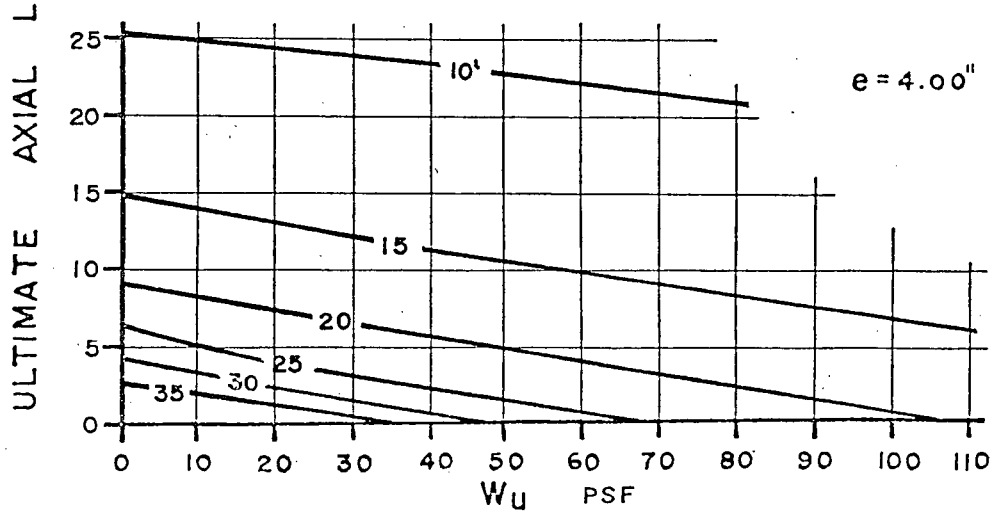
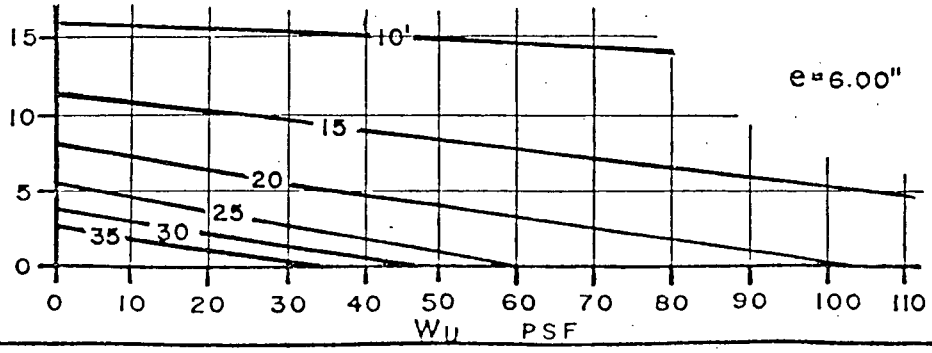








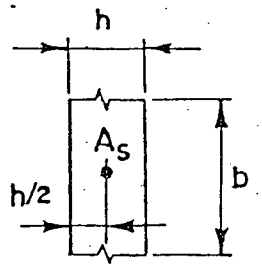
CONCRETE WALL PANEL
AXIAL LOAD CAPACITY

$h = 5.5$ IN

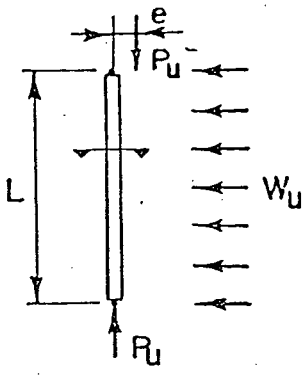
$A_s = .50$ IN^2/FT

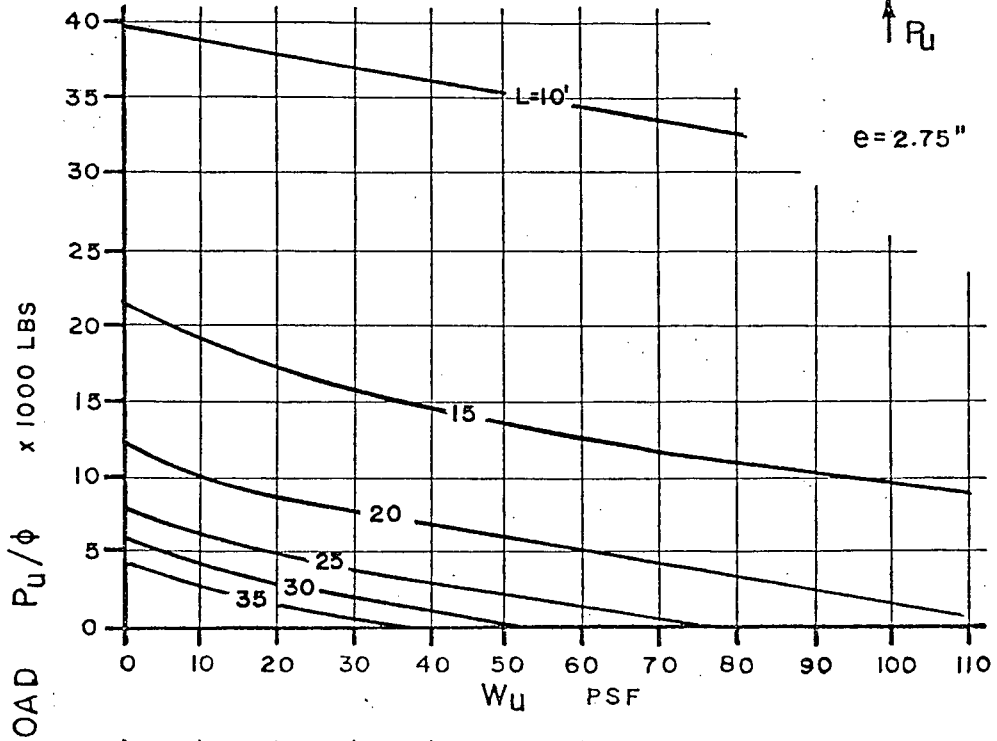
$f'_c = 4000$ PSI

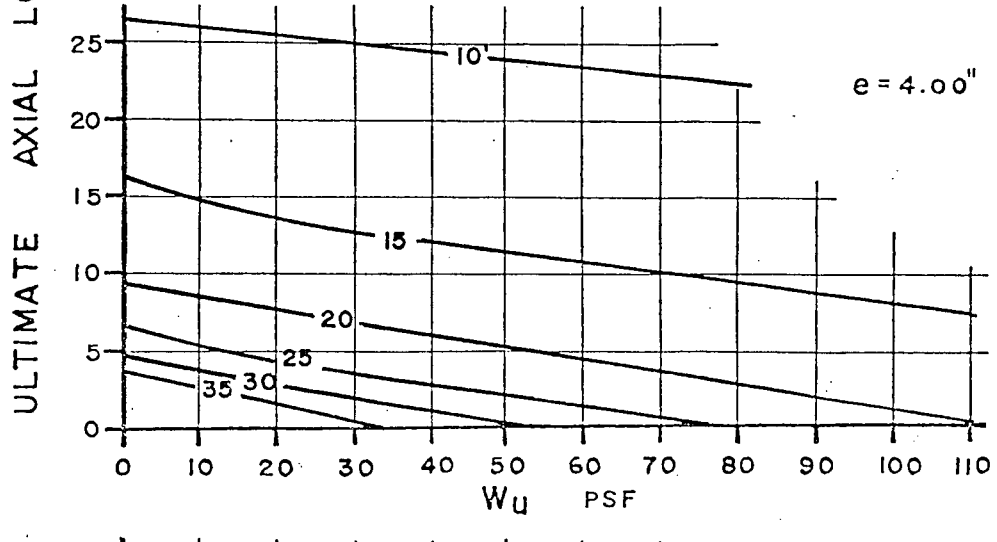
$f_y = 60000$ PSI

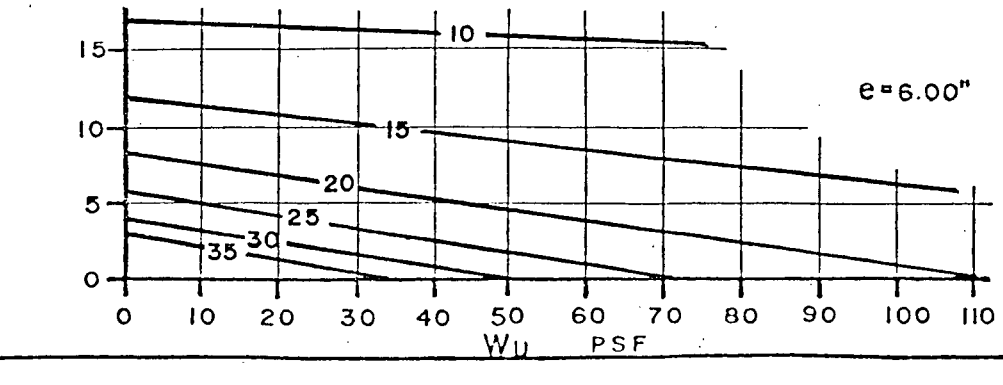


SECTION









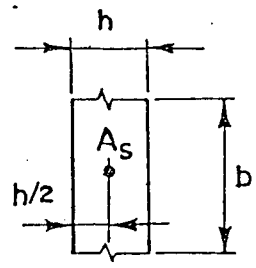
CONCRETE WALL PANEL
AXIAL LOAD CAPACITY

$h = 5.5$ IN

$A_s = .55$ IN^2/ FT

$f'_c = 4000$ PSI

$f_y = 60000$ PSI



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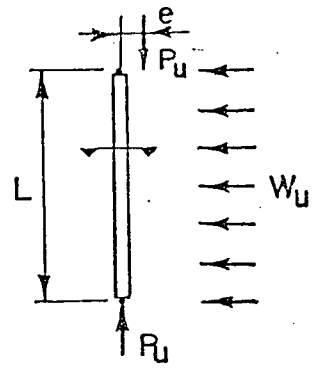
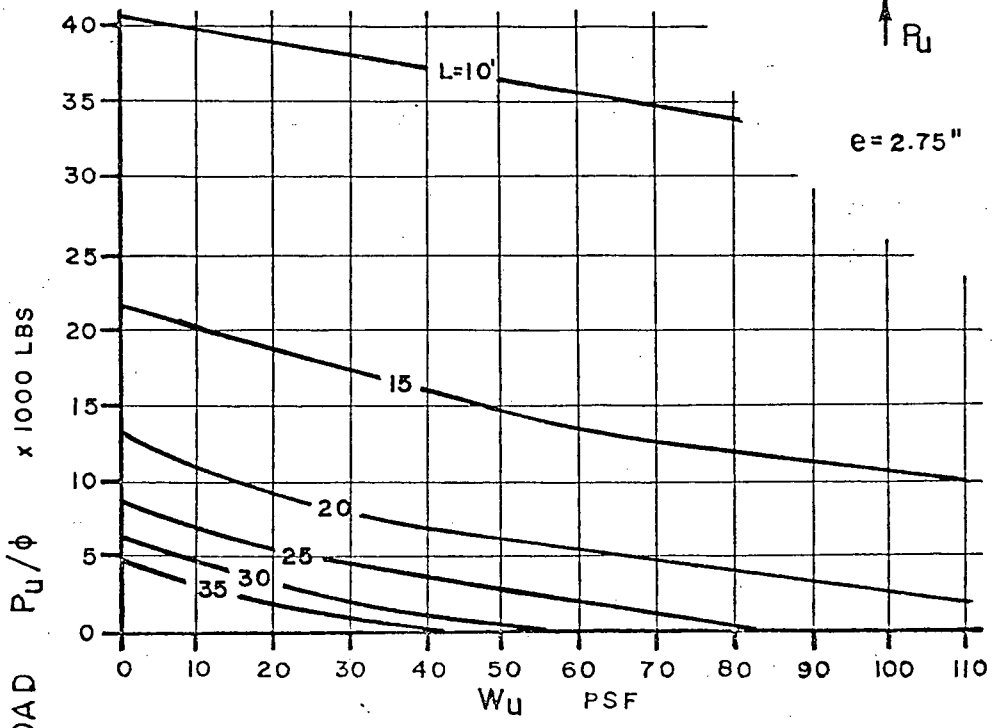
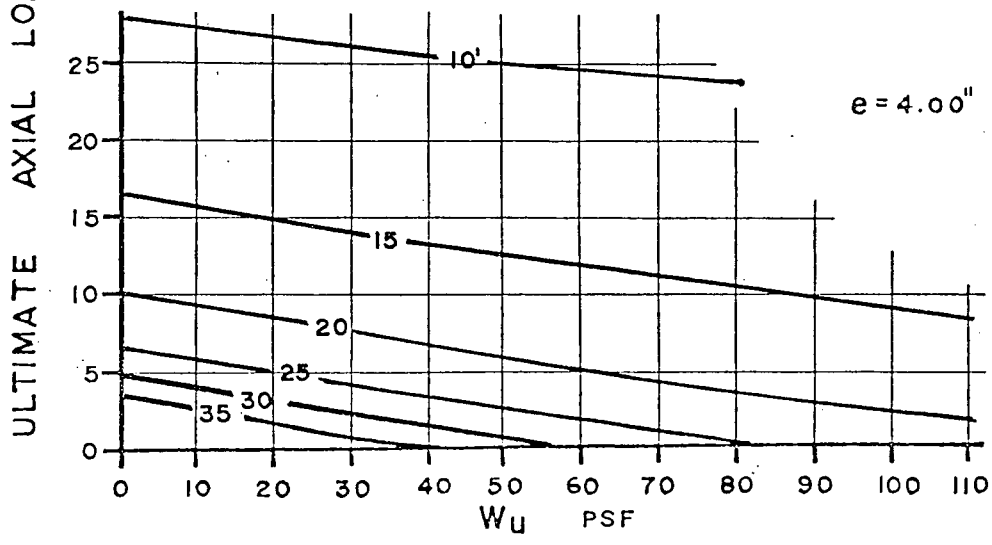
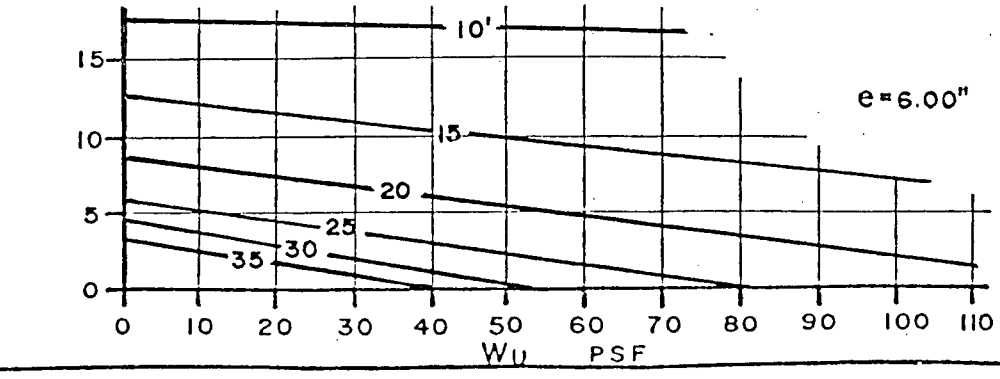








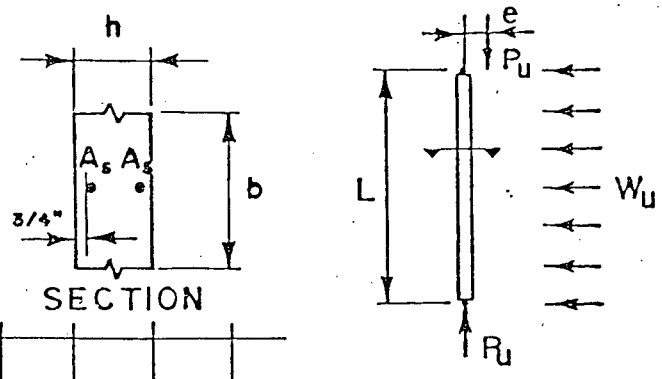
CONCRETE WALL PANEL AXIAL LOAD CAPACITY

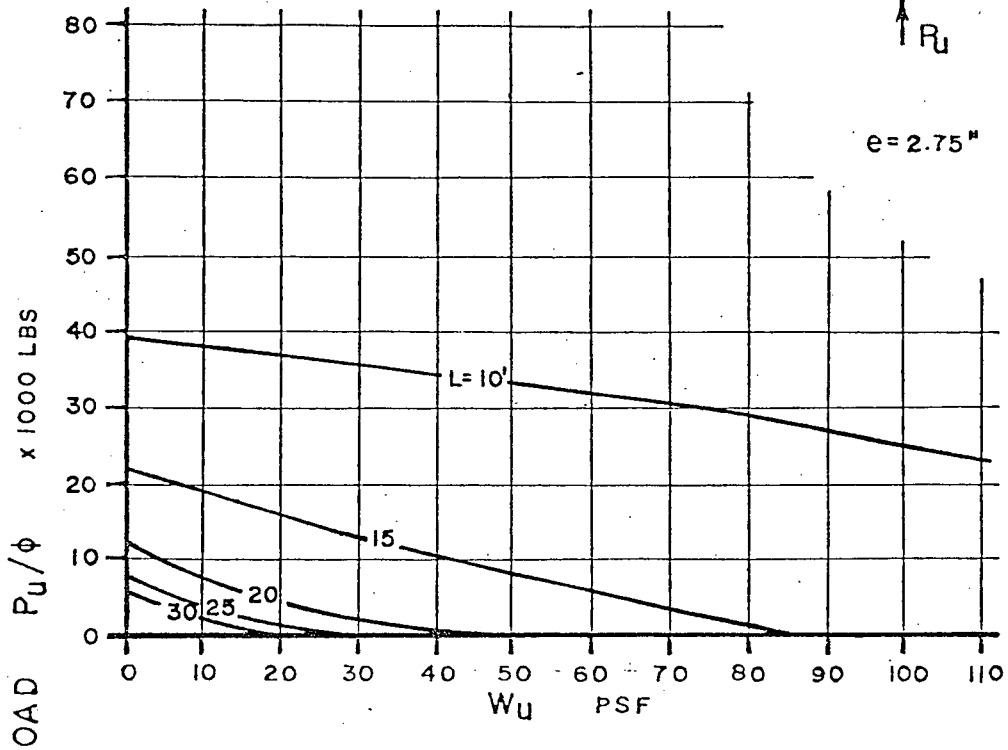
$h = 5.5$ IN

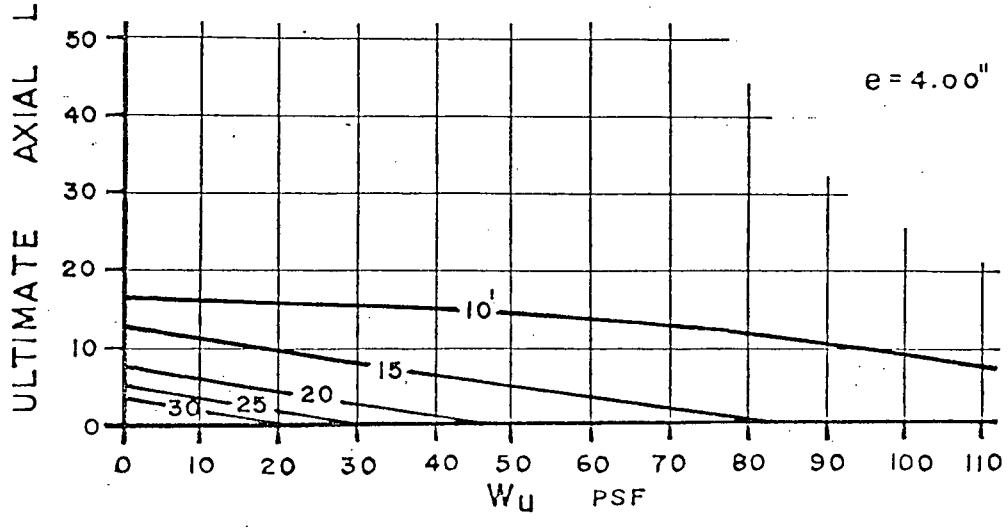
$A_s = .10$ IN^2/FT

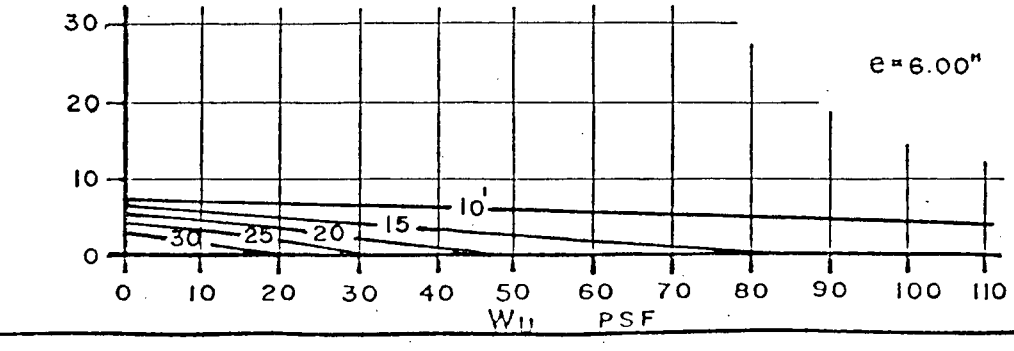
$f'_c = 4000$ PSI

$f_y = 60000$ PSI









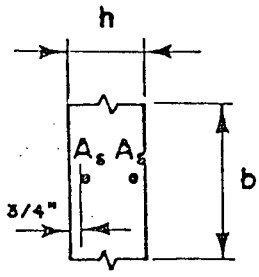
CONCRETE WALL PANEL AXIAL LOAD CAPACITY

$h = 5.5$ IN

$A_s = .15$ IN^2/FT

$f'_c = 4000$ PSI

$f_y = 60000$ PSI



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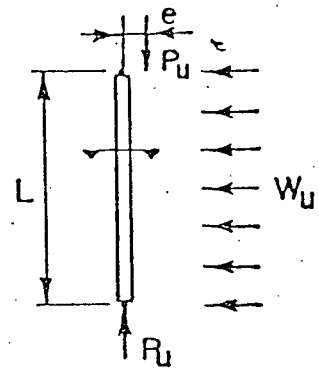
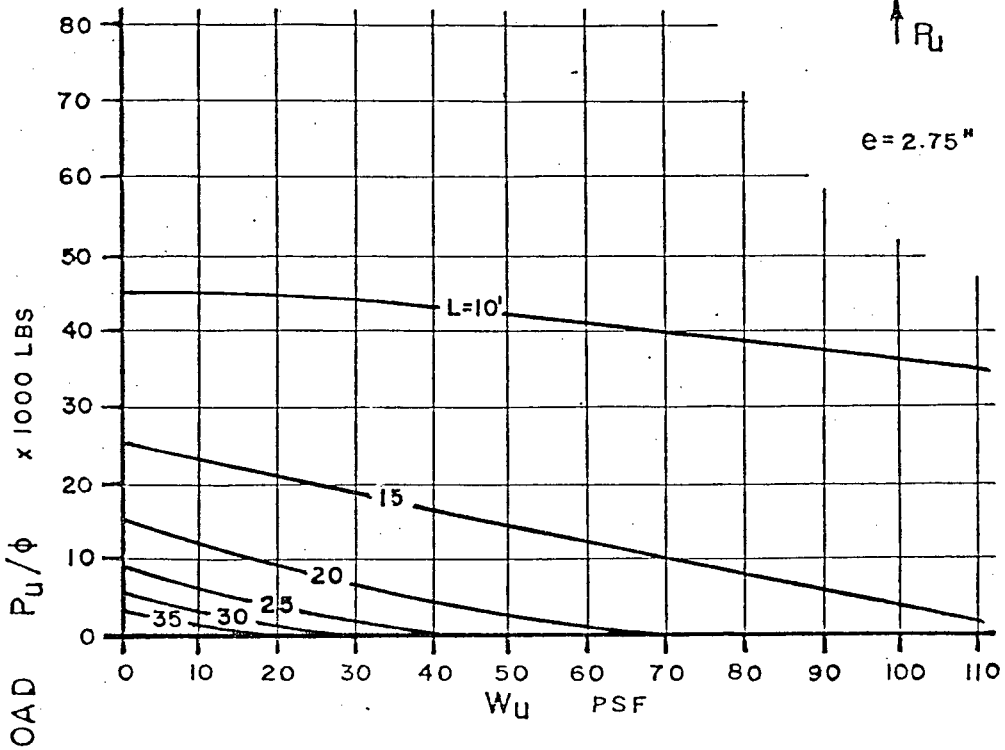
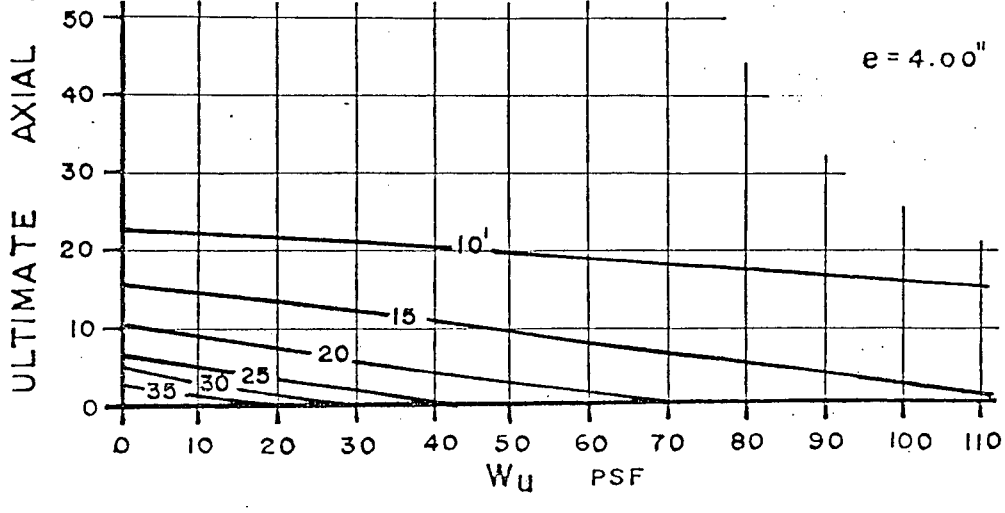
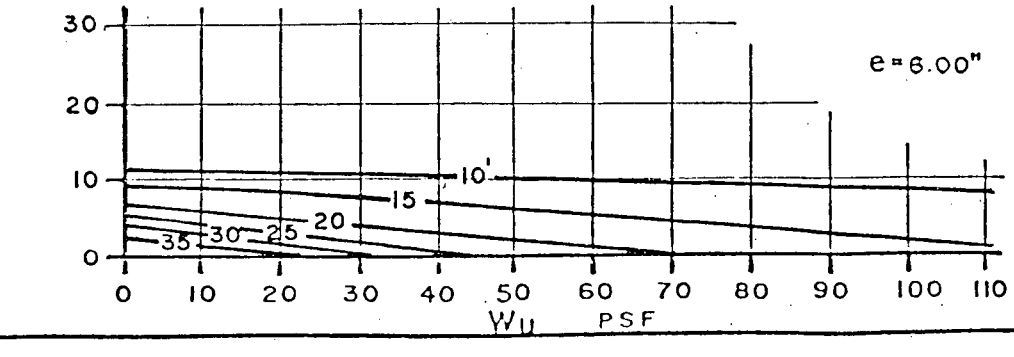








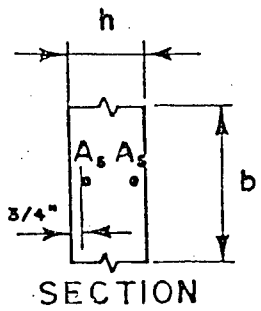
CONCRETE WALL PANEL AXIAL LOAD CAPACITY

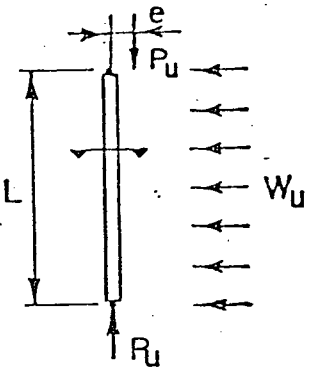
$h = 5.5$ IN

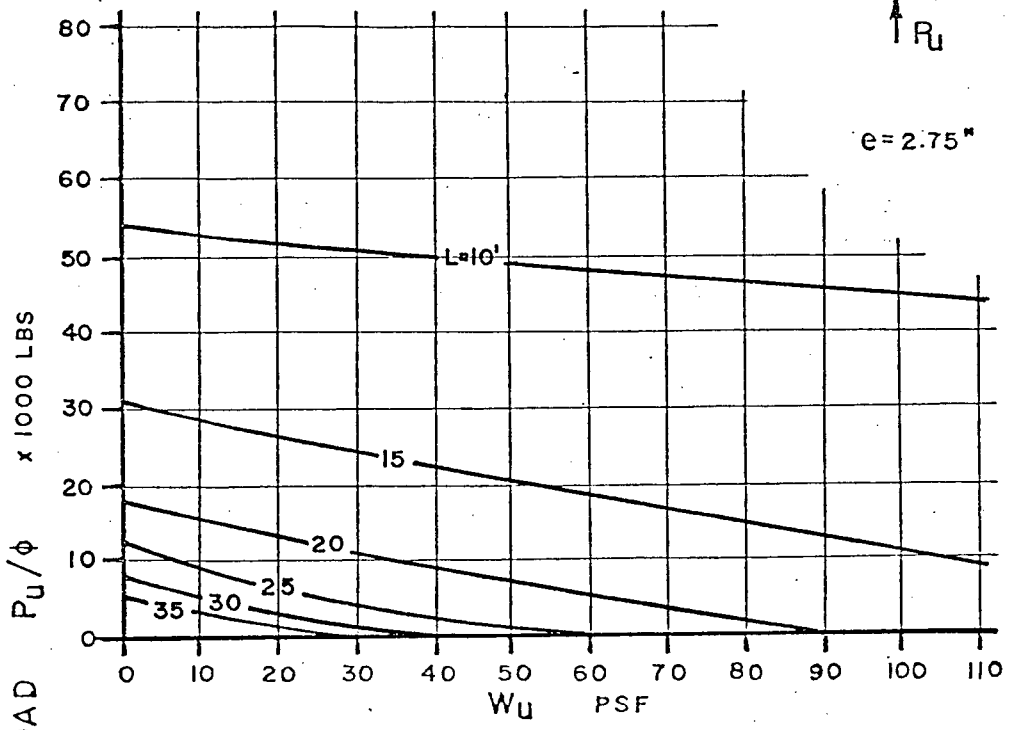
$A_s = .20$ IN^2/FT

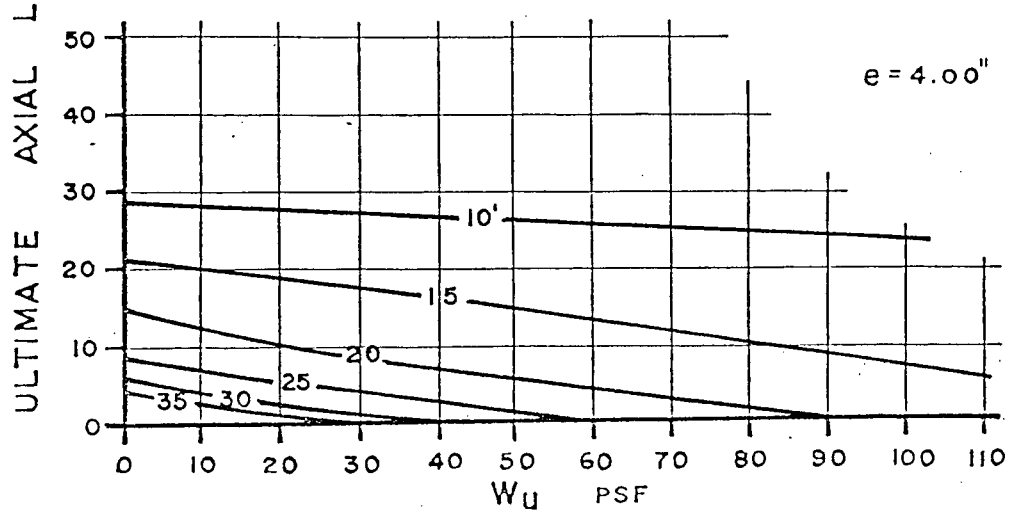
$f'_c = 4000$ PSI

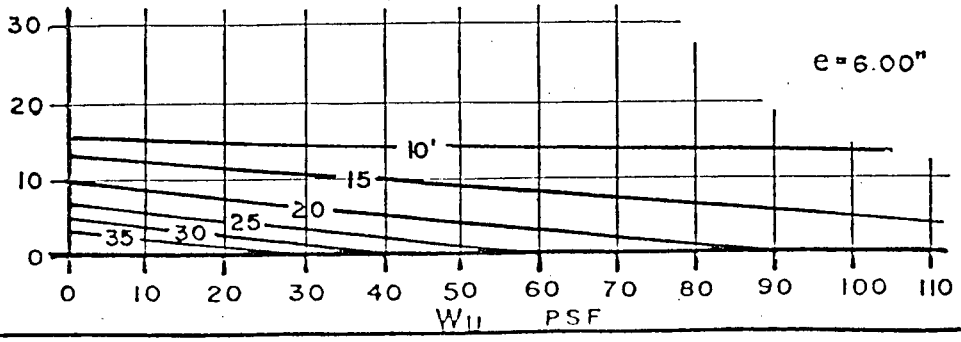
$f_y = 60000$ PSI











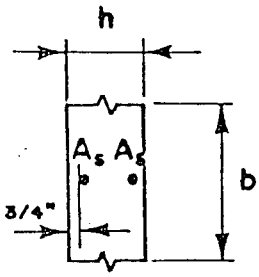
CONCRETE WALL PANEL
AXIAL LOAD CAPACITY

$h = 5.5$ IN

$A_s = .25$ IN^2/FT

$f'_c = 4000$ PSI

$f_y = 60000$ PSI



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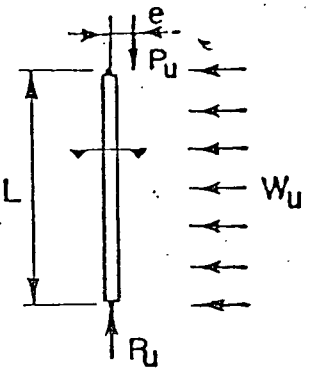
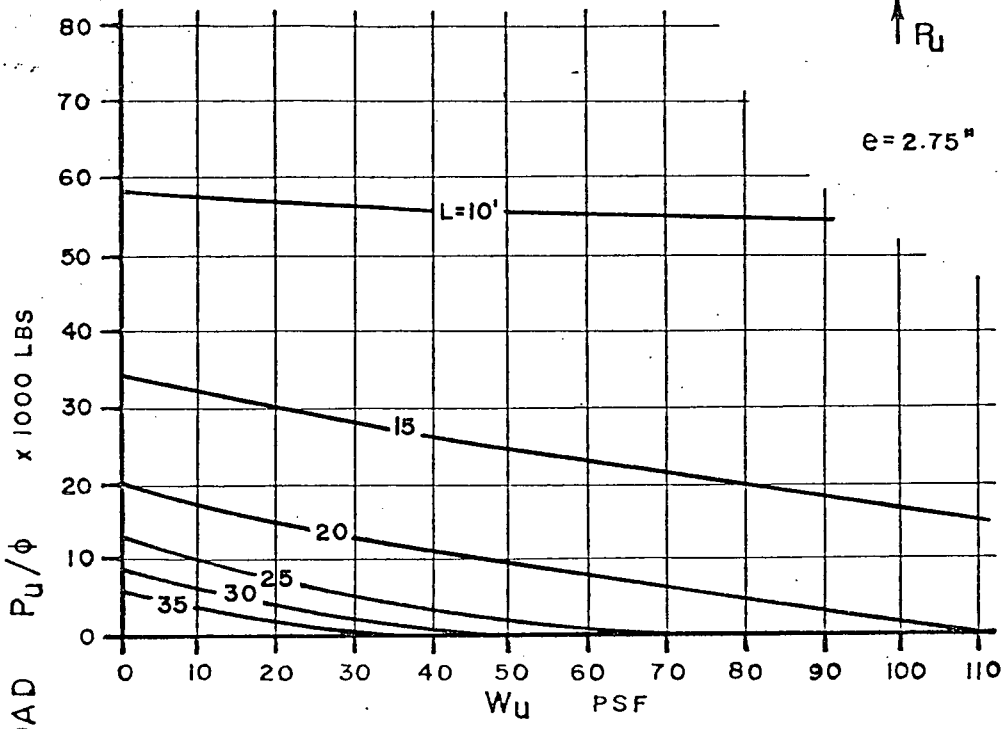
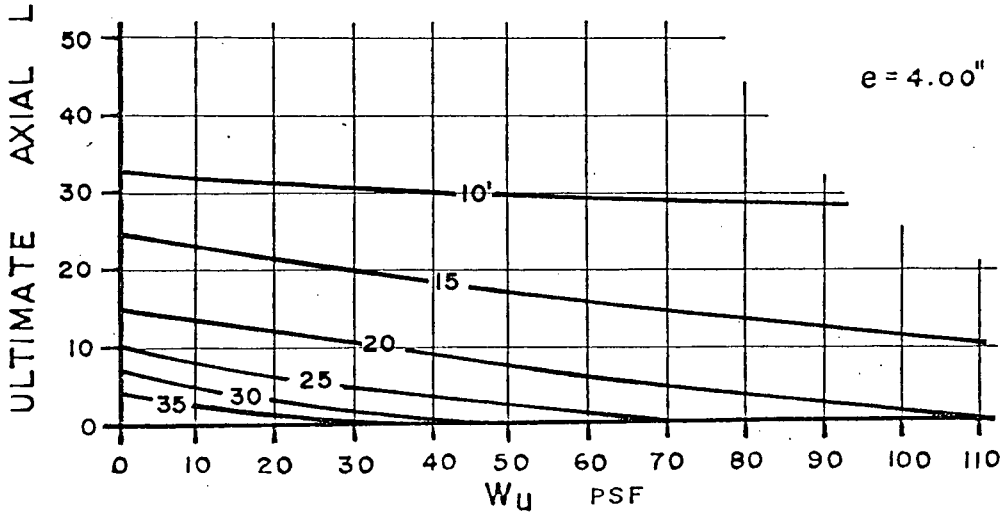
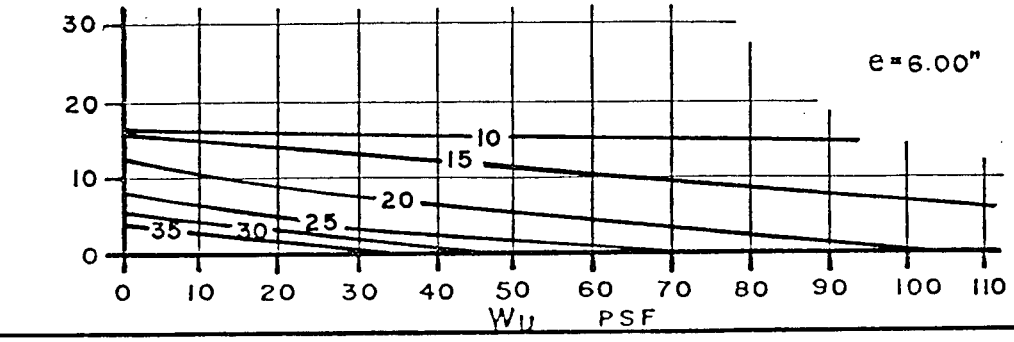








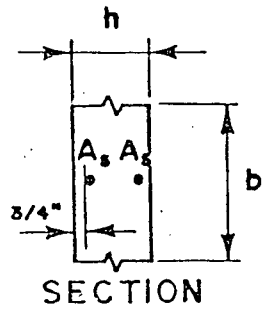
CONCRETE WALL PANEL
AXIAL LOAD CAPACITY

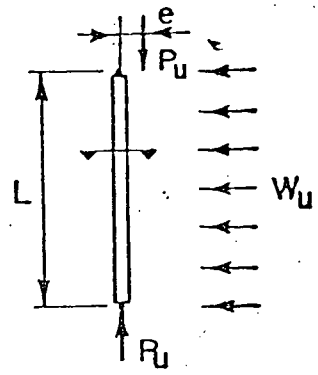
$h = 5.5$ IN

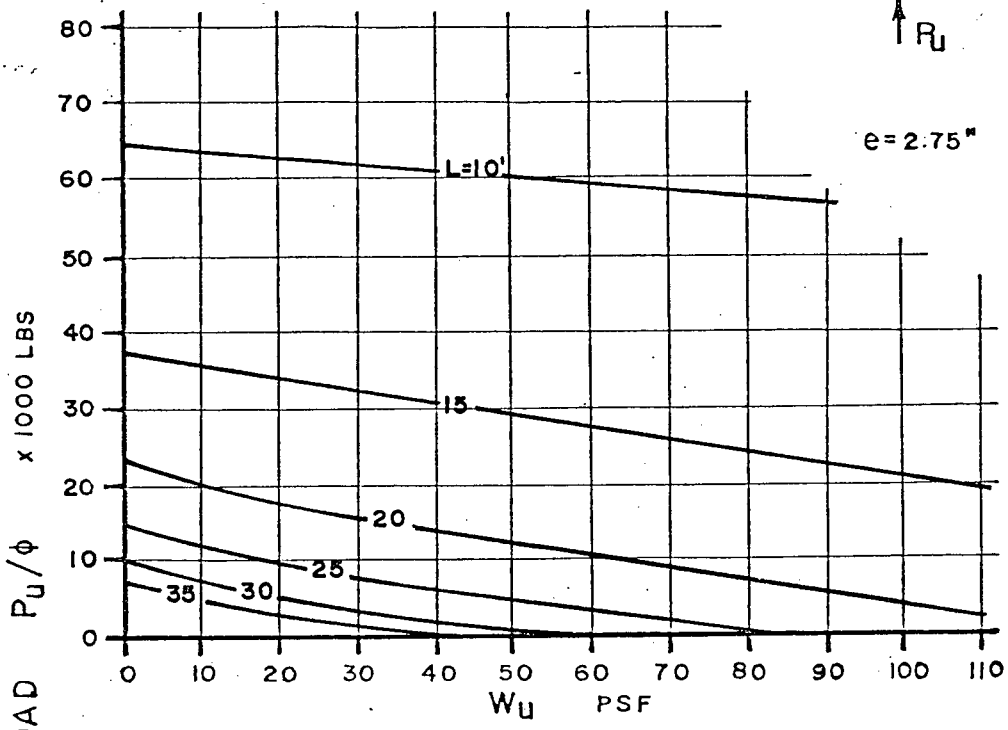
$A_s = .30$ IN^2/FT

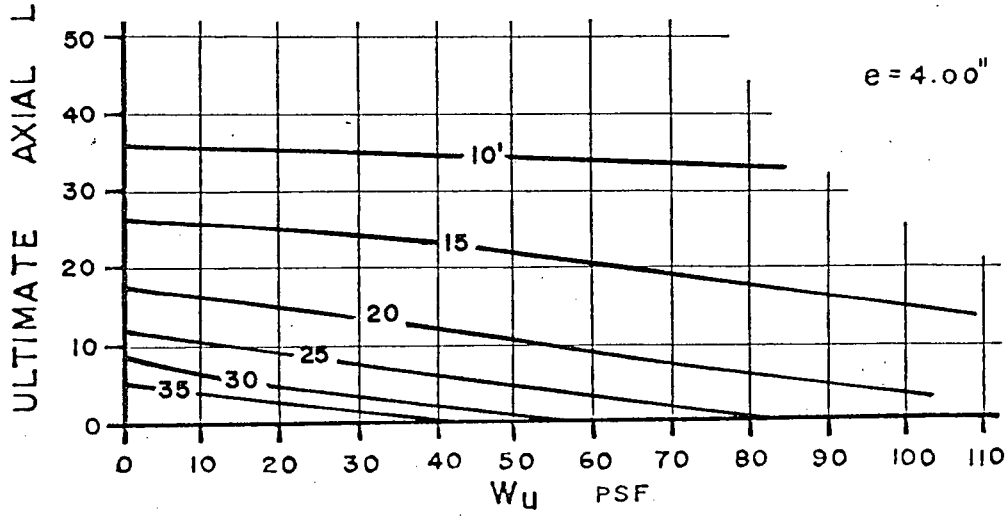
$f'_c = 4000$ PSI

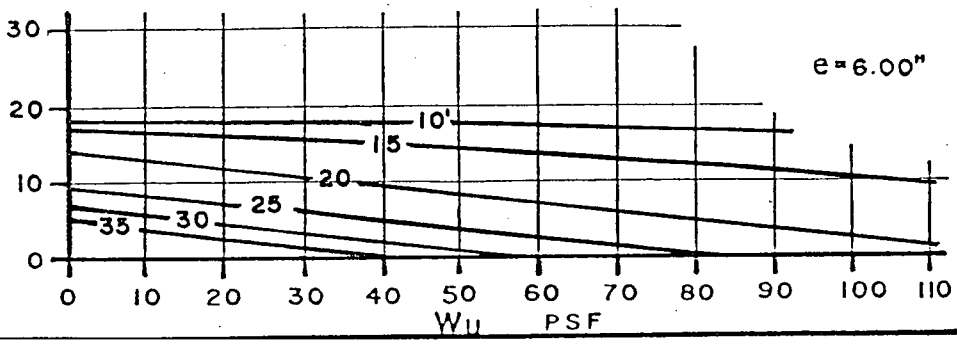
$f_y = 60000$ PSI











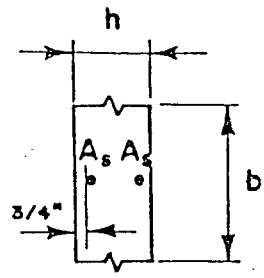
CONCRETE WALL PANEL
AXIAL LOAD CAPACITY

$h = 5.5$ IN

$A_s = .35$ IN^2/FT

$f'_c = 4000$ PSI

$f_y = 60000$ PSI



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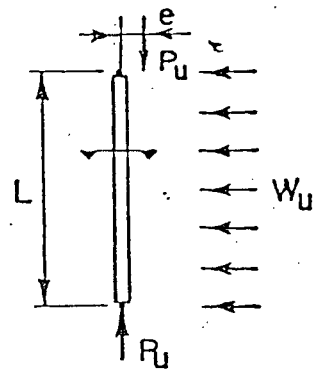
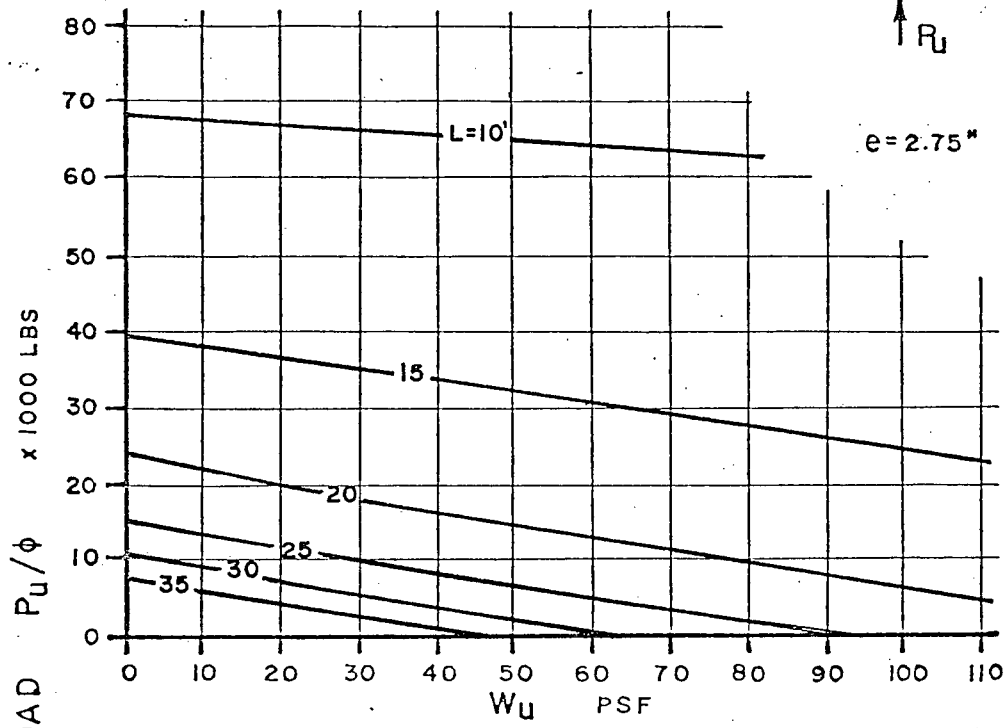
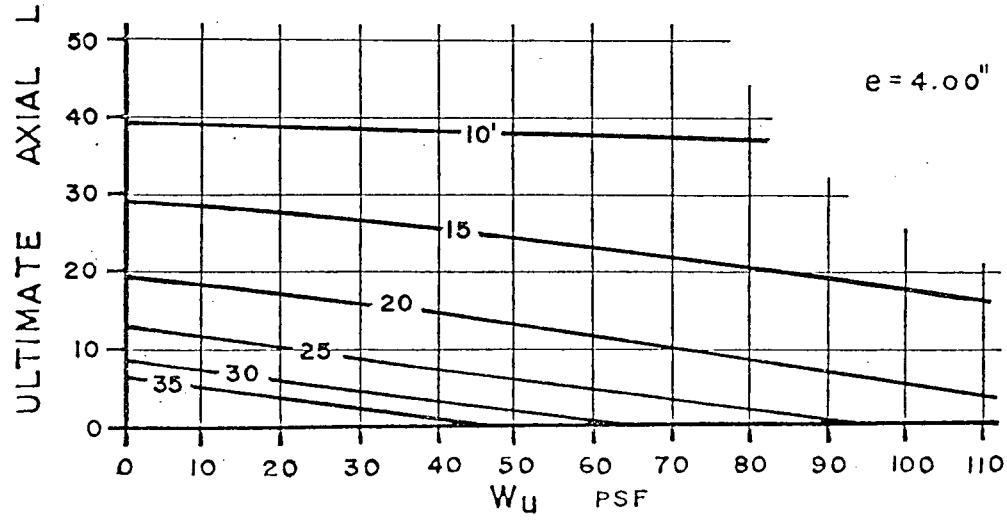
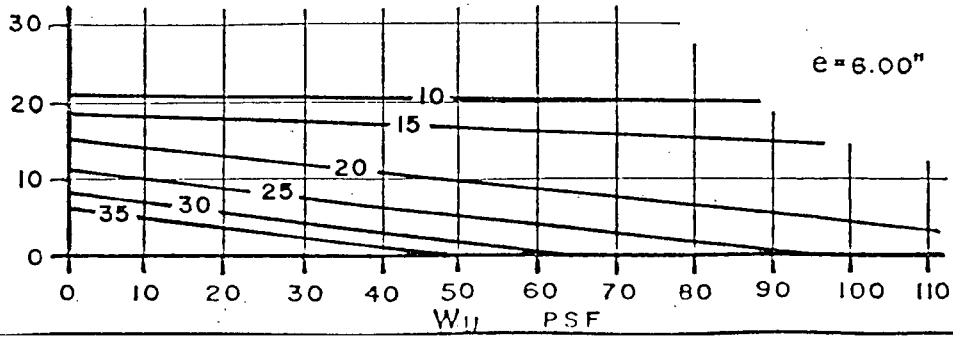








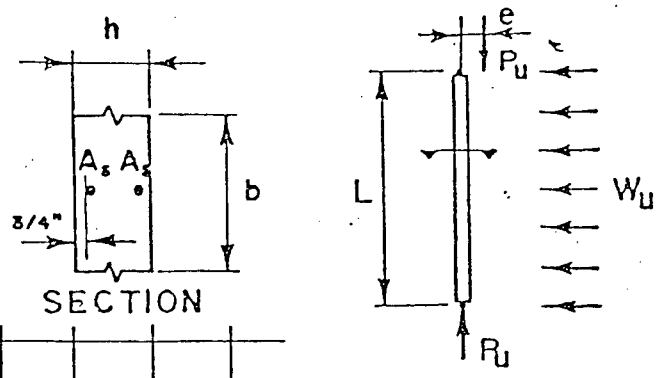
CONCRETE WALL PANEL
AXIAL LOAD CAPACITY

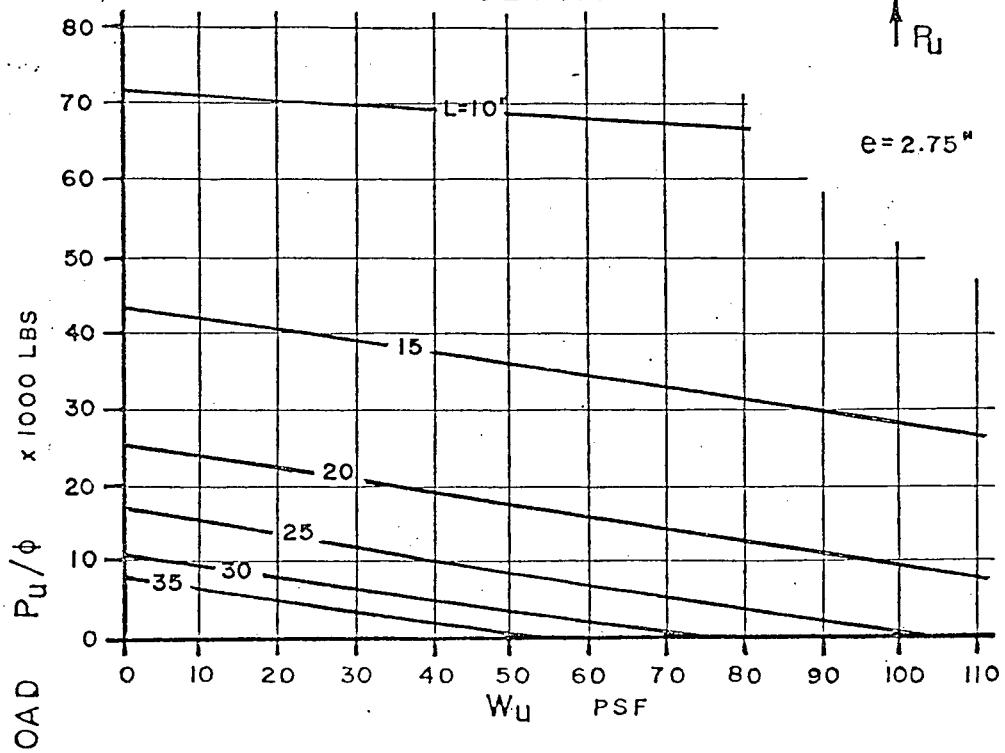
$h = 5.5$ IN

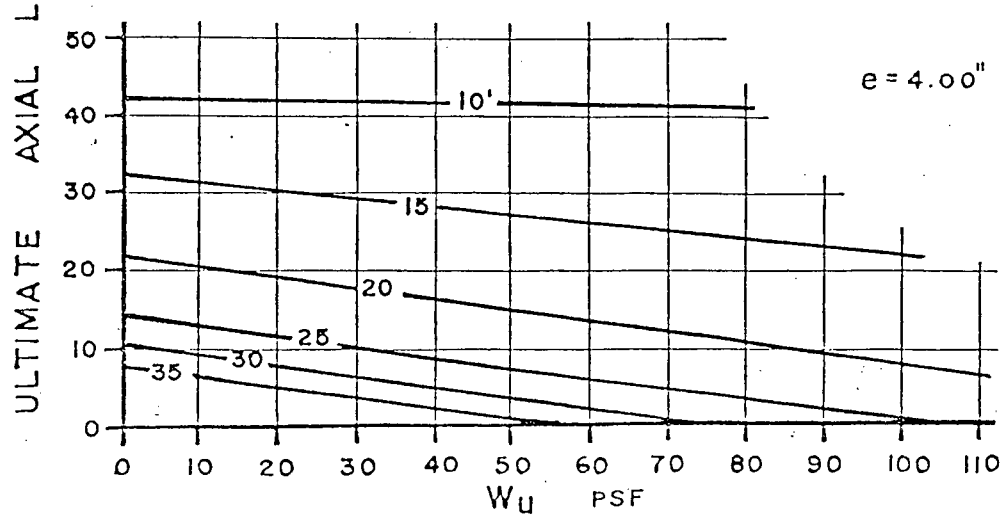
$A_s = .40$ IN^2/FT

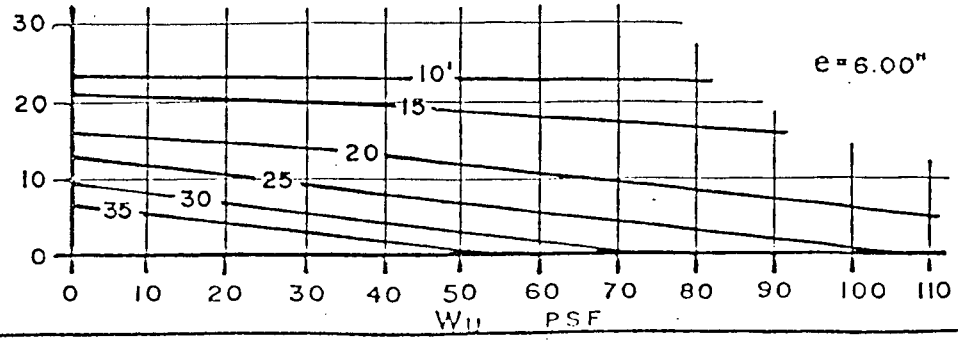
$f'_c = 4000$ PSI

$f_y = 60000$ PSI









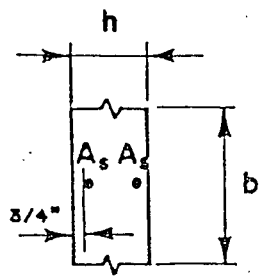
CONCRETE WALL PANEL AXIAL LOAD CAPACITY

$h = 5.5$ IN

$A_s = .45$ IN^2/FT

$f'_c = 4000$ PSI

$f_y = 60000$ PSI



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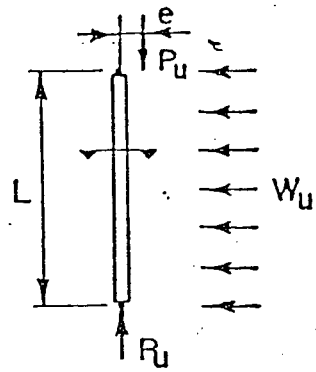
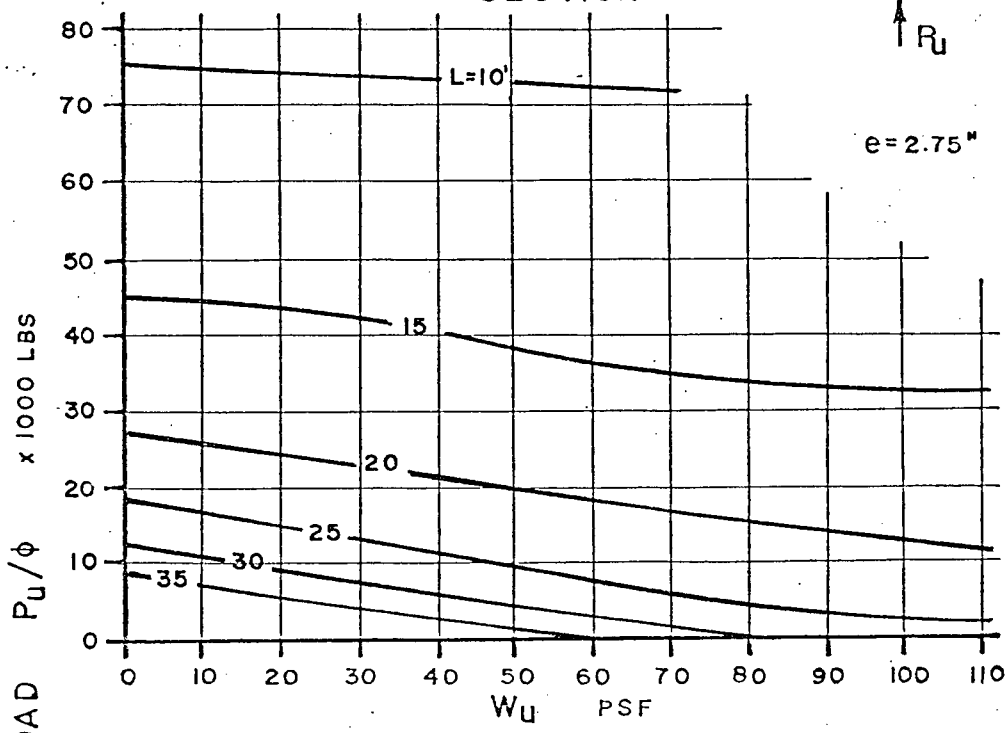
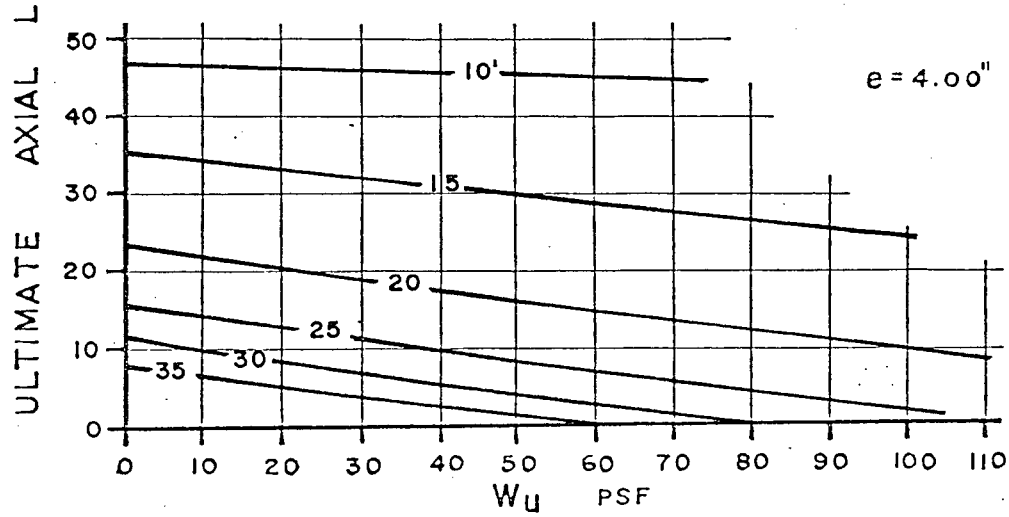
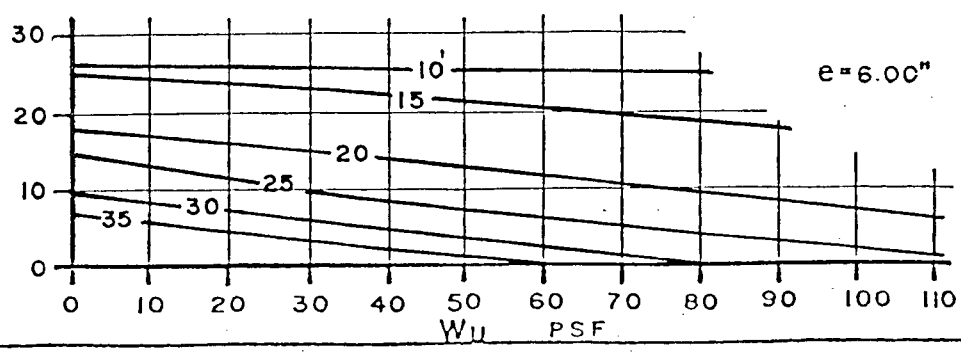








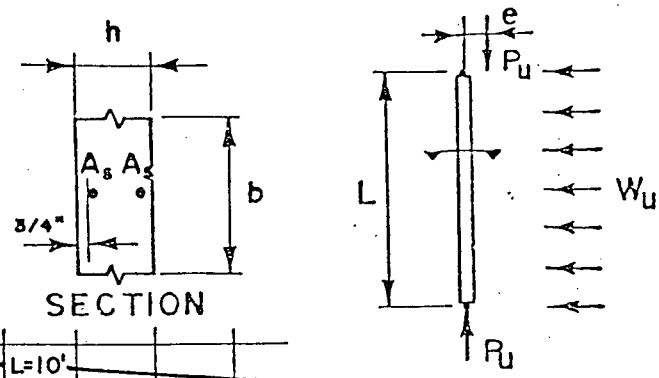
CONCRETE WALL PANEL AXIAL LOAD CAPACITY

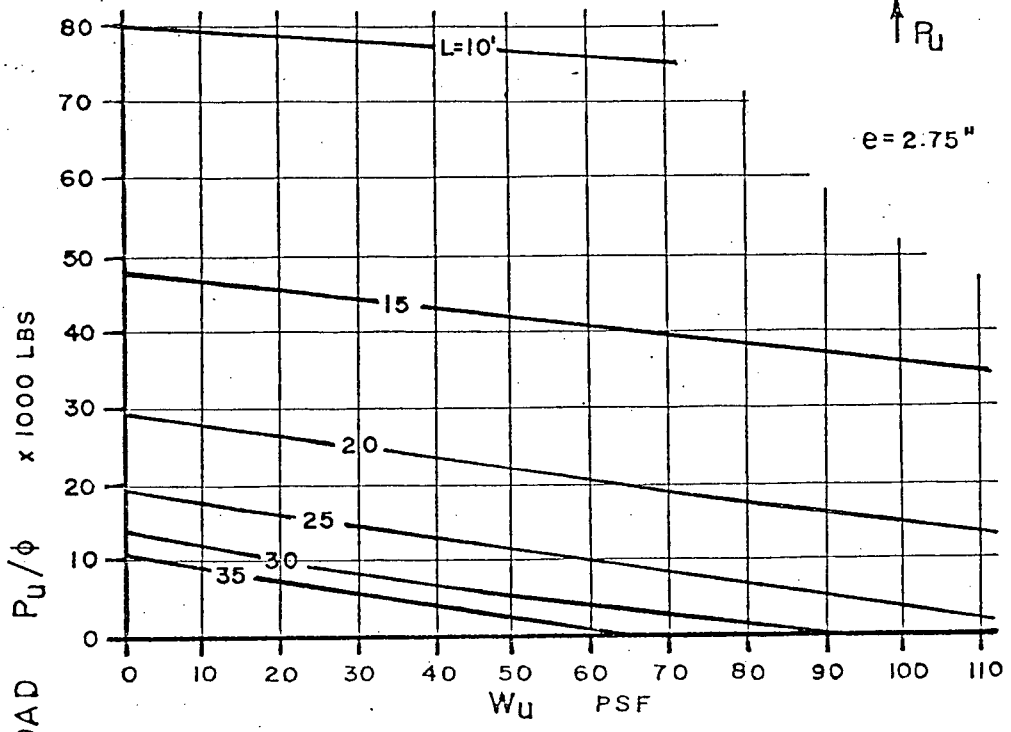
$h = 5.5$ IN

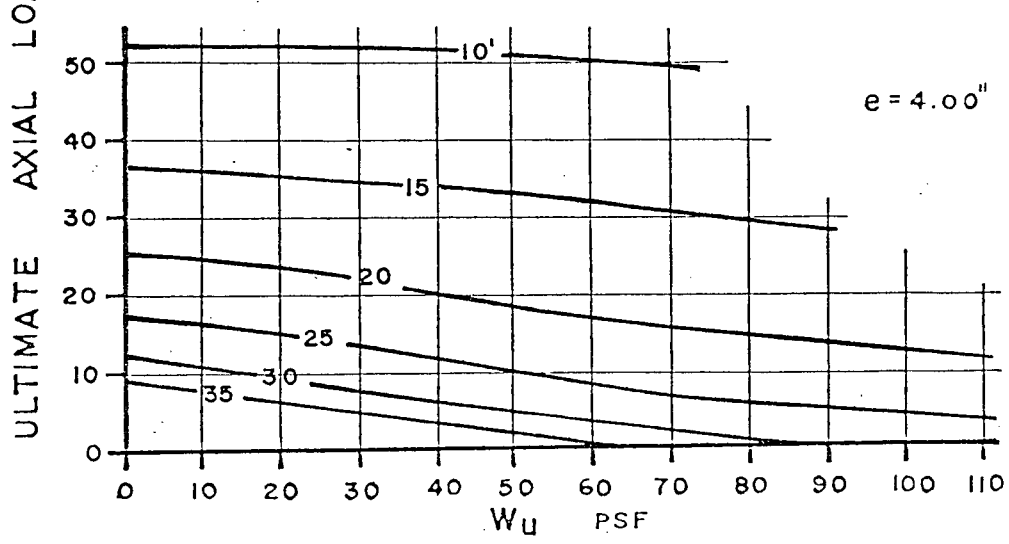
$A_s = .50$ IN^2/FT

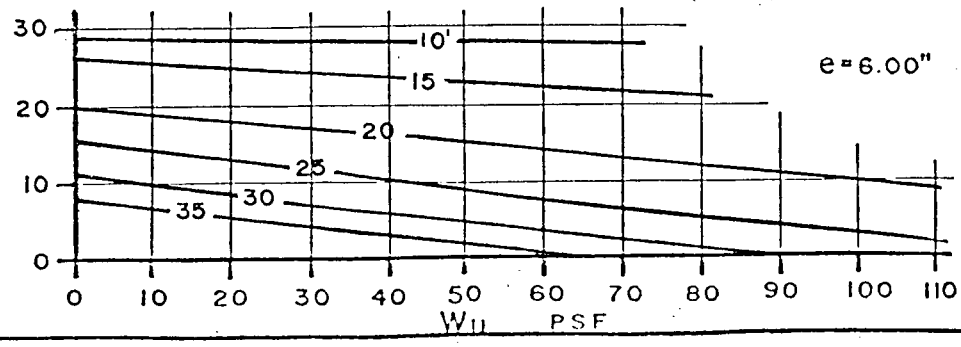
$f'_c = 4000$ PSI

$f_y = 60000$ PSI









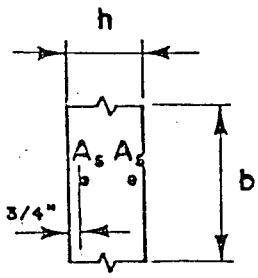
CONCRETE WALL PANEL AXIAL LOAD CAPACITY

$h = 5.5$ IN

$A_s = .55$ IN^2/FT

$f'_c = 4000$ PSI

$f_y = 60000$ PSI



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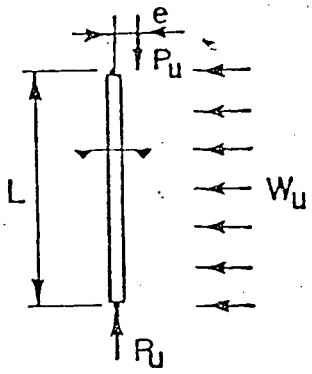
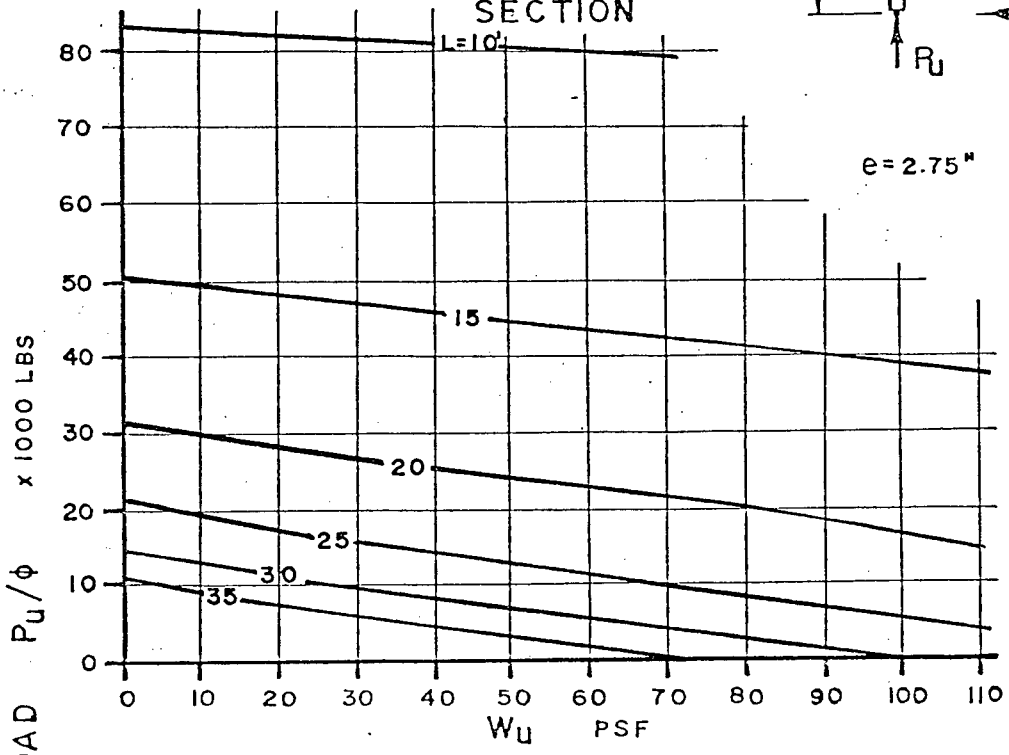
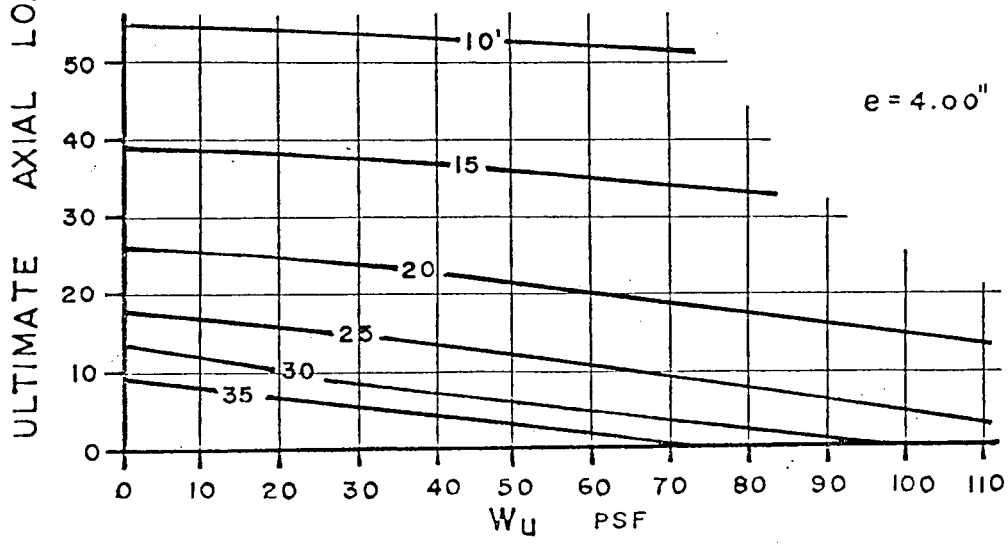
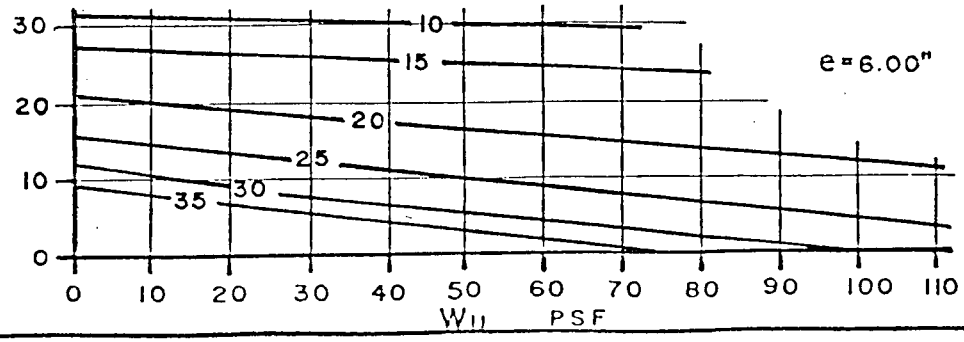








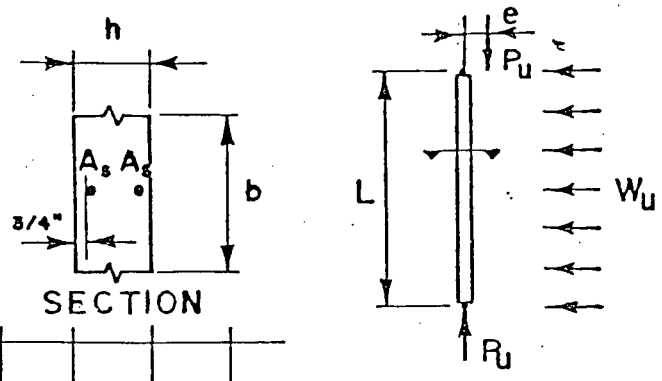
CONCRETE WALL PANEL
AXIAL LOAD CAPACITY

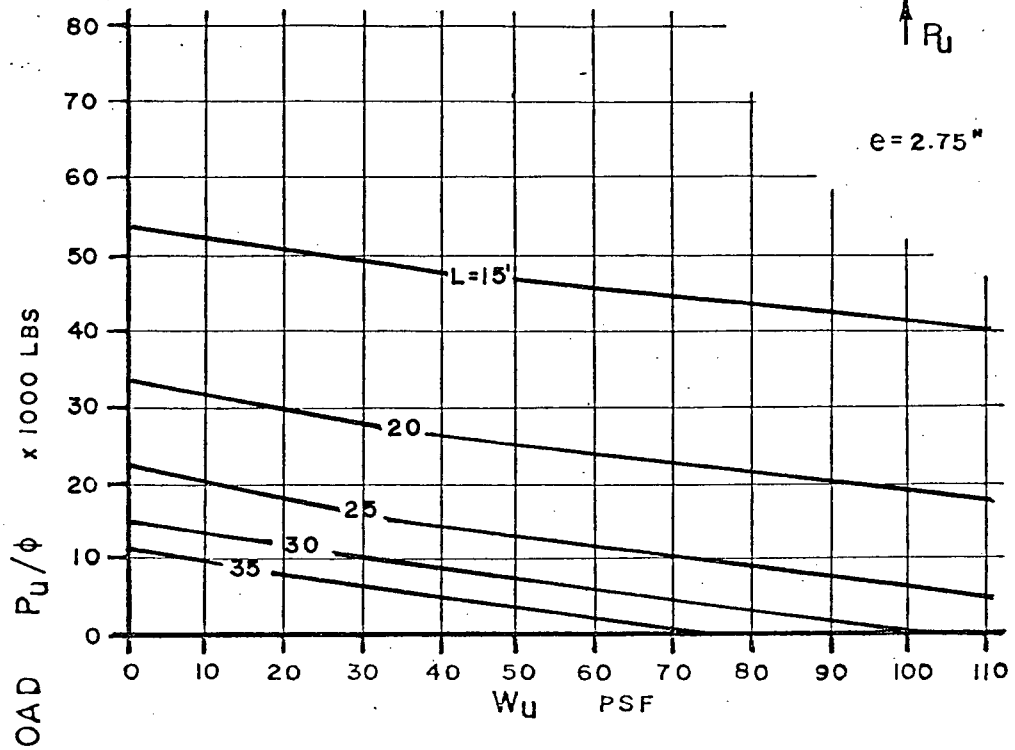
$h = 5.5$ IN

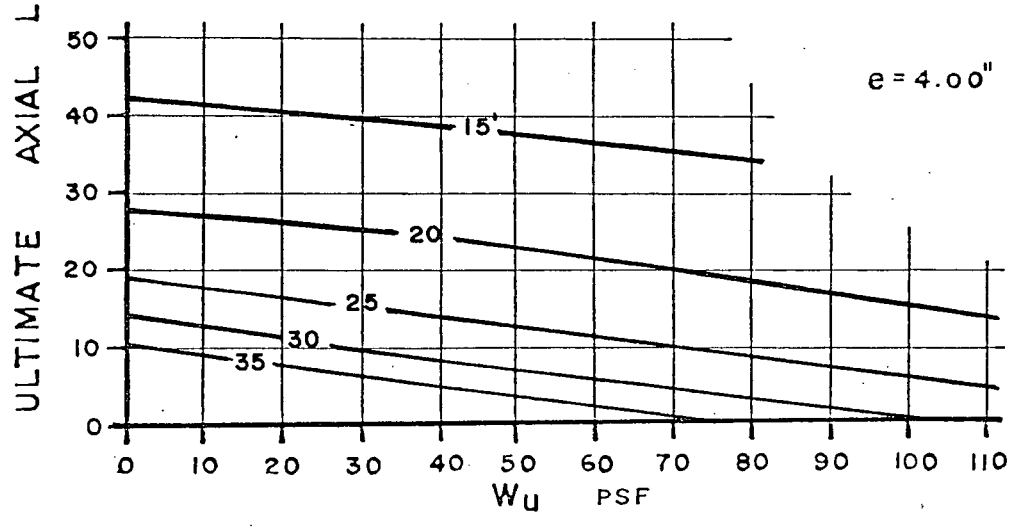
$A_s = .60$ IN^2/FT

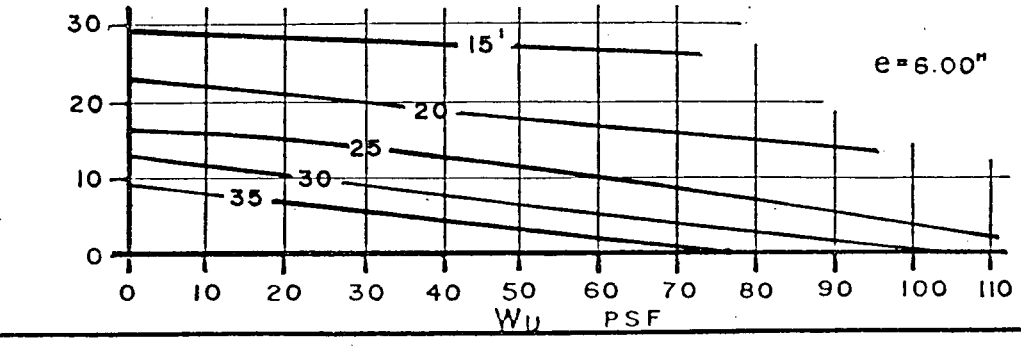
$f'_c = 4000$ PSI

$f_y = 60000$ PSI









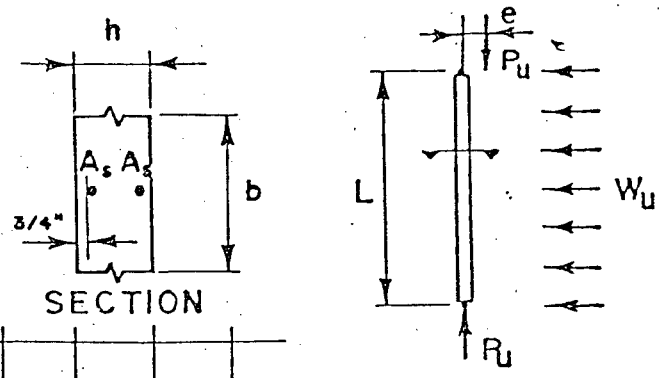
CONCRETE WALL PANEL
AXIAL LOAD CAPACITY

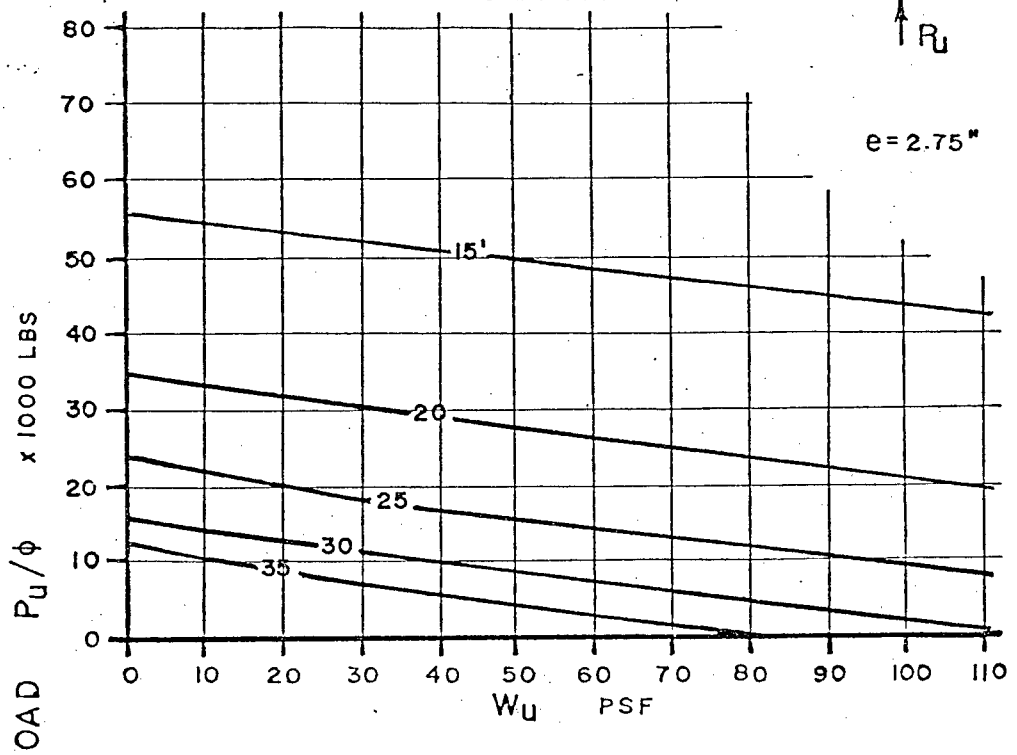
$h = 5.5$ IN

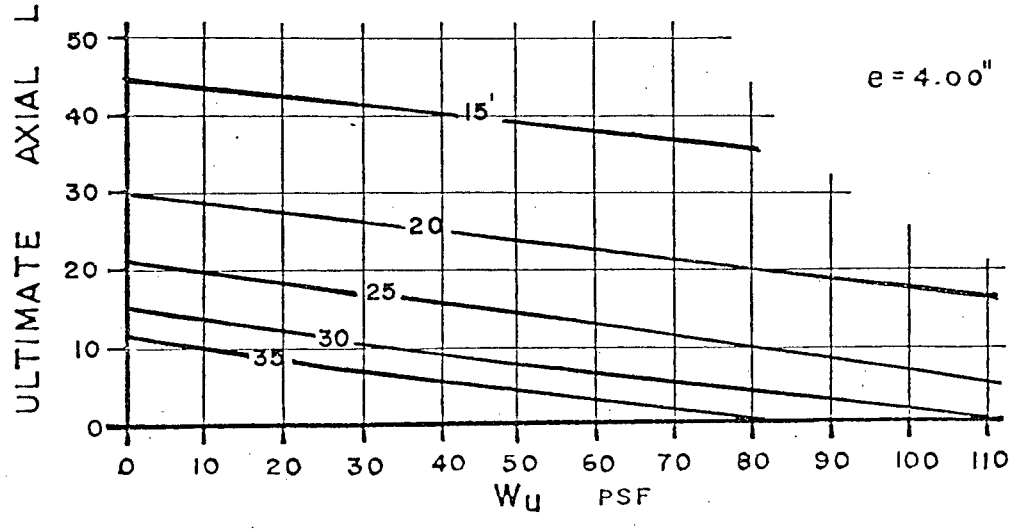
$A_s = .65$ IN^2/FT

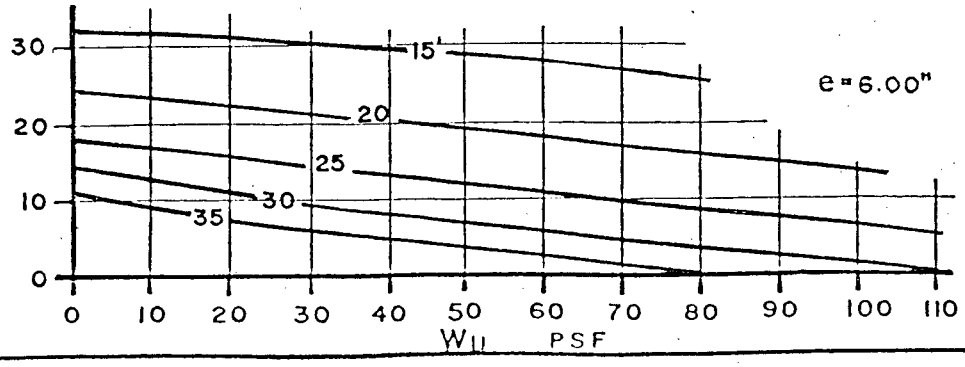
$f'_c = 4000$ PSI

$f_y = 60000$ PSI









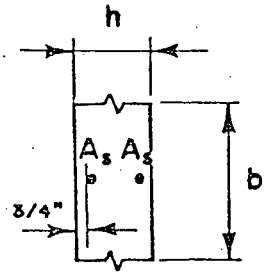
CONCRETE WALL PANEL
AXIAL LOAD CAPACITY

$h = 5.5$ IN

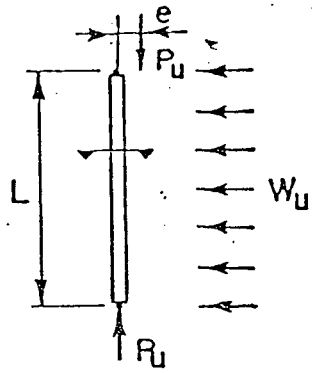
$A_s = .70$ IN^2/FT

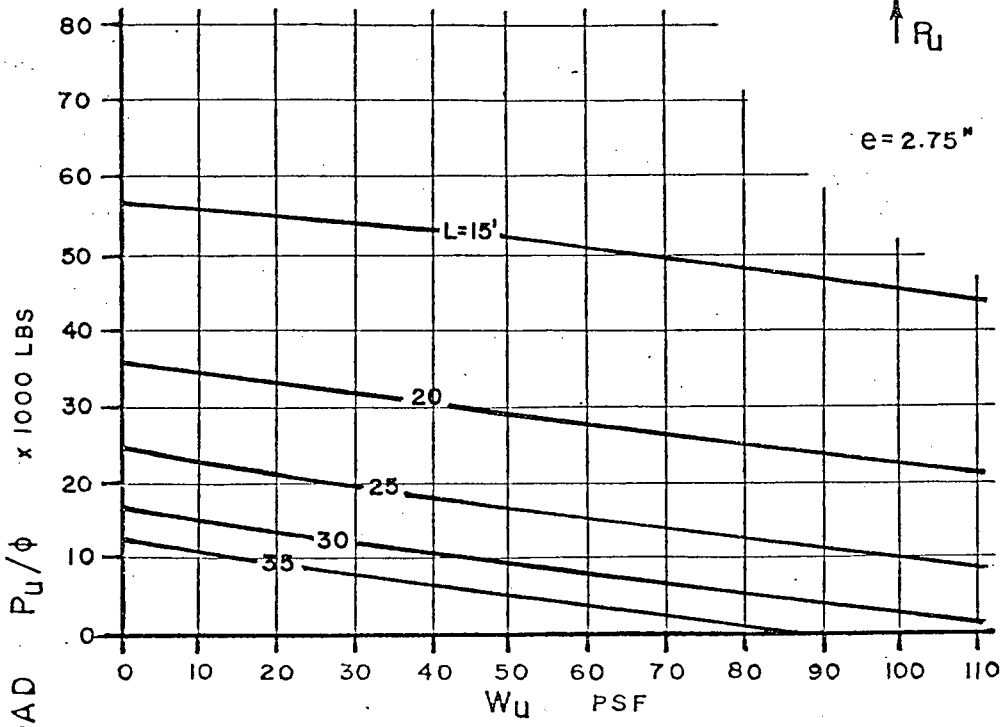
$f'_c = 4000$ PSI

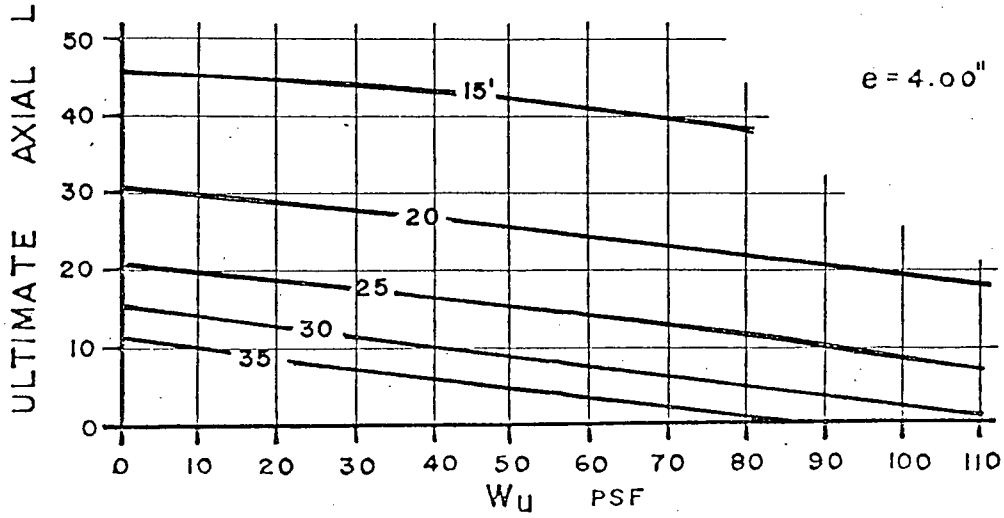
$f_y = 60000$ PSI

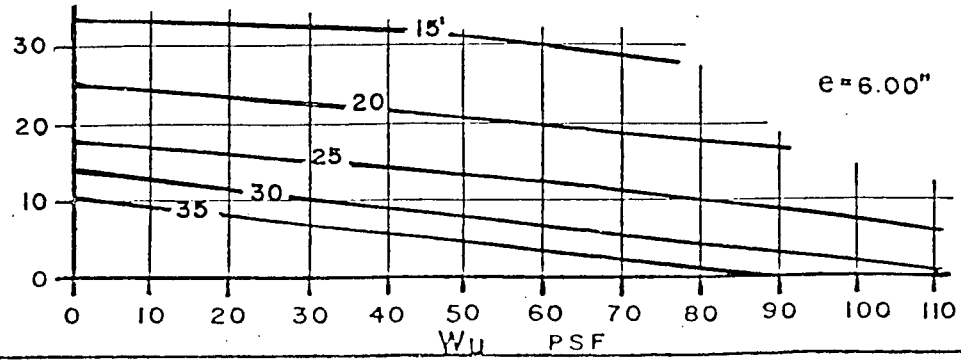


SECTION





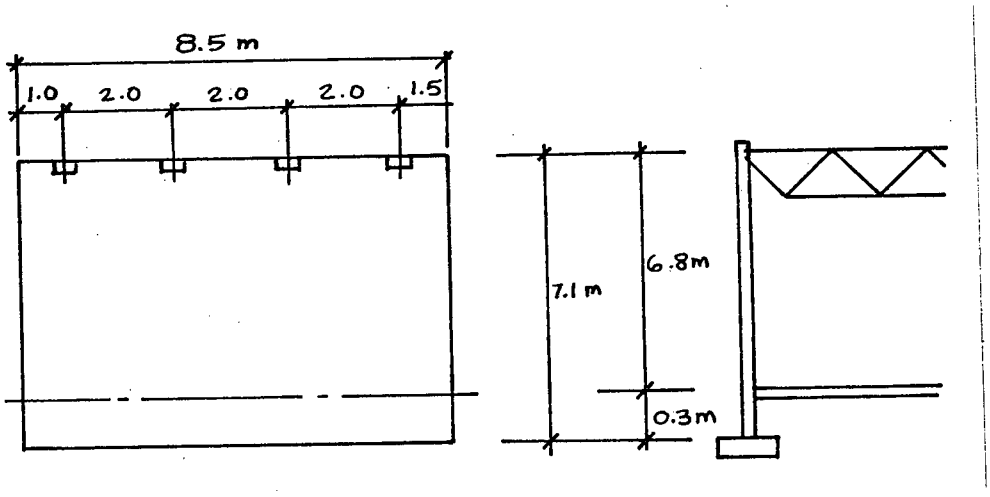




APPENDIX E

DESIGN EXAMPLES

DESIGN EXAMPLE #1
SIMPLE PANEL, JOIST LOADING



Given:

f'c = 25 MPa (normal wgt.)
fy = 400 MPa

Panel thickness h = 140 mm
Panel supported at roof joist and floor levels.

Joist reac. R = 7.5D + 15.0L = 22.5 kN
Wind load +W = 0.55 x (2 x .7 + .3) = 0.94 kN/m^2
-W = 0.55 x (2 z .7 - .3) = 0.61

Assume:

Joist load acts like U.D.L.
P = 22.5/2.0 = 11.25 kN/m (3.75D + 7.50L)
Load ECC = 70 mm for - wind
0 mm for + wind
Init. def'n. & creep Yo = 20 mm @ mid height

Span = 6.8 m, pinned-pinned support
Reinf. in centre of section d = 70 mm

Critical load case: dead plus +ve wind

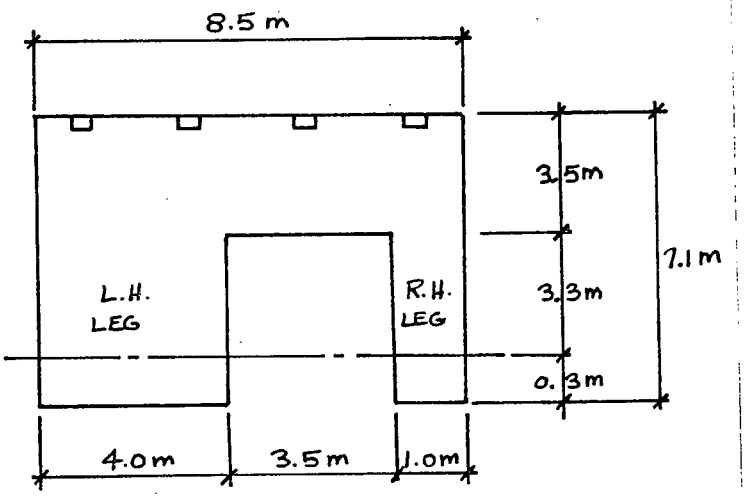
Axial load at mid height
P_u = 1.4 x (3.75 + 24 x .140 x 6.8/2) = 21.2 kN/m
Maximum applied moment
M_a = 1.7 x 0.94 x 6.8^2/8 + 21.2 x 0.020 = 9.66 kN-m/m

From chart A5, A_s = 800 mm^7/m

For P_u = 21.2 kN/m, L = 6.8 m
Max. M_a = 13.0 kN-m/^{7}m O.K.

Provide #15 @ 250 mm o/c, A_s = 800 mm^7/m

DESIGN EXAMPLE #2
PANEL WITH OPENING



Given:

Same span and loading as Example #1.

Truck door opening as shown.

R.H. Leg:
Assume 1.0 m wide strip carried loads from 1/2 width of opening.

Axial load at mid height:

$$P_u = 1.4 \times (3.75 + 24 \times .140 \times 6.8/2) \times \frac{1.75 + 1.0}{1.0} = 58.3 \text{ kN/m}$$

Maximum applied moment:

$$M_a = 9.66 \times 2.75 = 26.57 \text{ kN-m/m}$$

From Chart B6 $A_s = 1000 \text{ mm}^2/\text{m}$ E.F.

For $P_u = 58.3$ kN/m, $L = 6.8$ m

Max. $M_a = 27.5$ kn-m/m O.K.

Provide 5 #15 E.F., $A_s = 1000 \text{ mm}^2$

L.H. Leg - max. effective width = 1.75 m

$$P_u = 21.2 \times \frac{1.75 + 1.75}{1.75} = 42.4 \text{ kN/m}$$

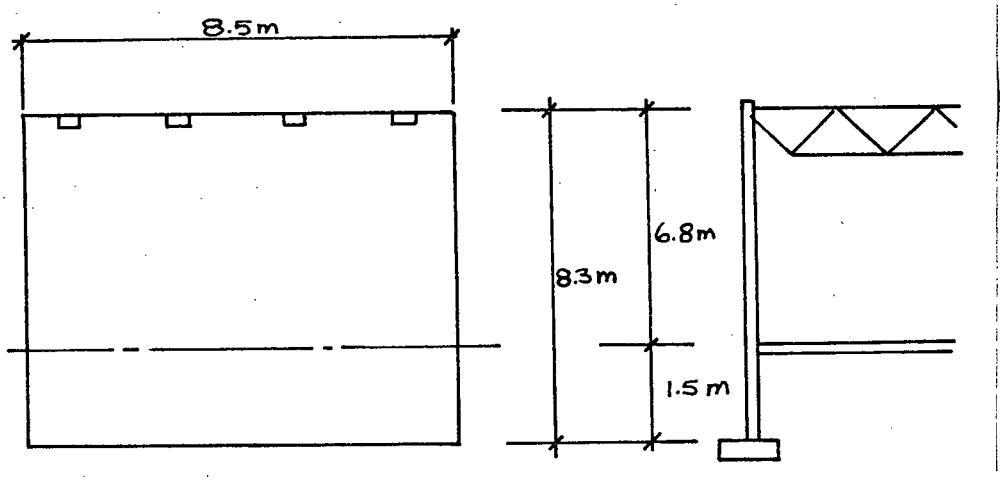
$$M_a = 9.66 \times \frac{1.75 + 1.75}{1.75} = 19.3 \text{ kN-m/m}$$

From Chart B5, $A_s = 800 \text{ mm}^2/\text{m}$ E.F.
Max. $M_a = 24$ kN-m/m O.K.

Provide #15 @ 250 mm E.F. For 1.75 m width
$A_s = 800 \text{ mm}^2/\text{m}$ E.F.

Remainder of L.H. Leg
#15 @ 400 mm E.F.

DESIGN EXAMPLE #3
EXTENSION BELOW FLOOR SLAB



Given:

Same as Example #1 with extension below floor slab. Effect of soil pressure and roof deflection ignored.

Assume:

Effective span = 0.85 x 6.8 = 5.78 m

For D + W loading:

P_u = 21.2 kN/m (from Example #1)

Assume: Max. span moment = max. support moment
= $WL^2/11.6$

$M_a = 1.7 \times 0.94 \times 6.8^2/11.6 + 21.2 \times .020 = 6.79$ kN-m/m

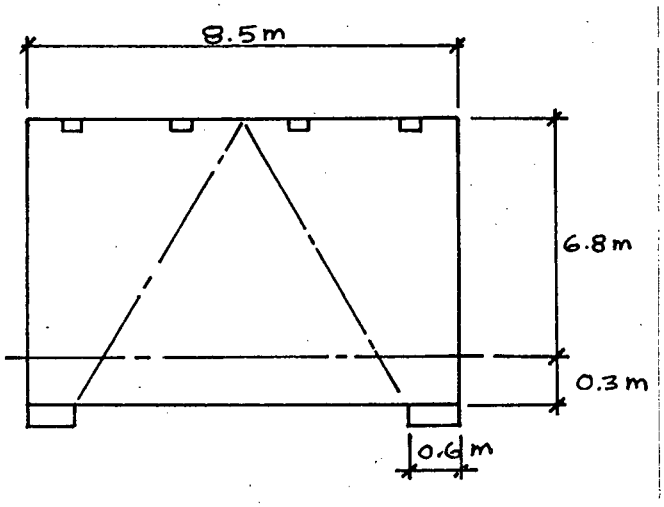
From Chart A2, $A_s = 400$ mm^2/m CTR

For $P_u = 21.2$ $L_u = 5.78$ m

Max. $M_a = 7.0$ kN-m/m O.K.

Provide #15 @ 400 mm o/c, $A_s = 500$ mm^2/m
in centre

DESIGN EXAMPLE #4
ISOLATED FOOTING



Given:

Same span and loading as Example #1.

Panel supported on pad footings as shown.

Assume distribution of vertical load into design strip above footing is linear from top to bottom.

Effective self-weight in design strip:

$$W_e = .140 \times 24 + \frac{3.75 + .140 \times 24 \times 7.1}{7.1} \left(\frac{8.5}{2 \times .6} - 1\right)$$

$$= 3.36 + 23.64 = 27.0 \text{ kN/m}^2$$

$$P_u = 1.4\ (27 \times 6.8/2 + 3.75) = 133.8 \text{ kN/m}$$

$$M_a = 9.66 \text{ kN-m/m} \quad \text{(from Example \#1)}$$

From Chart B5, $A_s = 800 \text{ mm}^2/\text{m}$ E.F.

For $P_u = 133.8$ $L = 6.8$ m

Max. $M_a = 11.0$ kN-m/m O.K.

Provide
#15 @ 250 mm E.F.
$A_s = 800 \text{ mm}^2/\text{m}$ E.F. over isolating footing