ENGINEERING HANDBOOK

structural design

section 6

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

PREFACE

This handbook is intended primarily for the use of Soil Conservation Service engineers. Much of the information will also be useful to engineers in other agencies and in related fields of work.

The aim of the handbook is to present in brief and usable form information on the application of engineering principles to the problems of soil and water conservation. While this information will generally be sufficient for the solution of most of the problems ordinarily encountered, full use should be made of other sources of reference material.

The scope of the handbook is necessarily limited to phases of engineering which pertain directly to the program of the Soil Conservation Service. Therefore, emphasis is given to problems involving the use, conservation, and disposal of water, and the design and use of structures most commonly used for water control. Typical problems encountered in soil and water conservation work are described, basic considerations are set forth, and all of the step-by-step procedures are outlined to enable the engineer to obtain a complete understanding of a recommended solution. These solutions will be helpful in training engineers and will tend to promote nation-wide uniformity in procedures. Since some phases of the field of conservation engineering are relatively new, it is expected that further experience may result in improved methods which will require revision of the handbook from time to time.

The handbook material has been prepared by M. M. Culp, Head of the Engineering Standards Unit of the Engineering Division, and his associates, Woody L. Cowan and Carroll A. Reese, under the general direction of the Engineering Council. The Council is made up of the regional engineers and the Chief of the Engineering Division at Washington. Under its direction the needs of engineers in all parts of the country have been considered and are reflected in the subject matter selected, the method of presentation, and the organization of the different sections.

Many sources of information have been utilized in developing the material. Original contributions and verbatim use of previously published materials are acknowledged in the text.

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A = effective tension area of concrete per bar;
= area;
= coefficient.

A_b = area of individual reinforcing bar.

a = height of vertical slab fixed on three edges;
= distance from end of beam to concentrated load;
= ratio of distance from end of beam to start of partial uniform load to total beam span.

b = width of compression face of member;
= breadth or width of vertical slab fixed on three edges.

b_c = breadth of conduit.

b_d = breadth or width of ditch or trench at top of conduit.

C = coefficient $[k(1 - k)] \div 2$ (see ES-1);
= ratio of load intensities, $q_2 \div q_1$, (see ES-3);
= ratio of stiffnesses, $K_s \div K_w$, (see ES-28 and ES-29).

C_c = coefficient (see page 6.2-22 and ES-22).

C_d = coefficient (see page 6.2-18 and ES-15).

D = nominal diameter of bar
= $k(1 - k^2)$ (see ES-3).

d = effective depth of section;
= distance between two concentrated loads (see ES-25).

d_b = nominal diameter of bar.

d_c = thickness of concrete cover to center of bar closest to extreme fiber.

E = $3 \div (1 + k)$ (see ES-3).

e = 2.7183 = base of Naperian logarithms;
= eccentricity;
= moment arm.

f_c = compressive stress in concrete.

f'_c = compressive strength of concrete.

f_s = stress in reinforcement.

f'_s = stress in compressive flexural reinforcement.

f_t = tensile stress in plain concrete.

f_y = yield strength of reinforcement.

H = height of wall;

= vertical distance from top of fill to top of conduit;

= vertical distance between center lines of top and bottom of conduit (see ES-28 and ES-29);

= horizontal component of force.

H_c = vertical distance from top of fill to top of conduit.

H_e = height of equal settlement.

h = vertical dimension or distance (see ES-4);

= vertical inside dimension of conduit (see ES-28 and ES-29).

i = angle of surcharge between fill slope and horizontal.

j = ratio used in reinforced concrete relations.

K = ratio of active lateral pressure to vertical pressure;

= a shape factor coefficient for wind loads.

K_s = stiffness of slab (see ES-28 and ES-29).

K_w = stiffness of wall (see ES-28 and ES-29).

k = radius of gyration;

= ratio of distance from end of beam to a point on the beam to total span length.

L = distance between center lines of vertical walls of rectangular conduit (see ES-28 and ES-29).

ℓ = horizontal inside dimension of conduit (see ES-28 and ES-29);

= length of beam.

ℓ_d = development length.

M = moment.

M_{ab} = moment at end "a" of member "ab."

M^F_{ab} = fixed end moment at end "a" of member "ab."

M^s_c = simple beam moment at point "c."

$M_{k\ell}$ = moment at distance $k\ell$ from end of beam.

M_x = $M_{k\ell}$ where $x = k\ell$.

m = (see ES-29).

N = direct force.

n = modular ratio of steel to concrete;

= (see ES-29).

P = resultant active force per foot of wall (see page 6.2-4);

= concentrated load.

P_H = horizontal component of P.

P_V = vertical component of P.

P_p = resultant passive force per foot of wall (see page 6.2-15).

p = intensity of pressure at a point;
= (see ES-28).

p_n = normal unit pressure on inclined surface.

psf = pounds per square foot.

psi = pounds per square inch.

q = intensity of load on a beam;
= (see ES-28).

R = resultant of a system of forces on loads;
= reaction.

R_A = reaction at end "A" of beam.

R_A^F = reaction at end "A" of fixed ended beam.

R_A^S = simple beam reaction at end "A."

S = shear.

U_A^F = algebraic sum of fixed end moments at joint "A."

u = bond stress.

V = vertical component of force;
= wind velocity;
= shear.

v = shear stress in concrete.

v_c = allowable shear stress taken by concrete.

V_A = shear at end "A" of structural member.

V_A^F = shear at end "A" of fixed ended member.

V_A^S = shear at end "A" of simply supported member.

$V_{k\ell}$ = shear at distance $k\ell$ from end of member.

W = total load on a beam.

W_c = total load per unit of length on a conduit.

w = specific weight of soil per unit volume;
= weight of equivalent fluid per unit volume.

x = coordinate length (usually horizontal).

y = coordinate length (usually vertical).

Z = factor associated with flexural crack width;
= angle of friction between soil and back of wall.

z = ratio of lengths or distances.

α = $(2K\mu') \div b_d$ (see page 6.2-17).

γ = unit weight of backfill (see ES-15 and ES-22).

δ = settlement deflection ratio (see ES-22).

θ = angle between horizontal and inclined surface;

= angle between horizontal and back face of wall.

μ = tangent of angle of internal friction in backfill.

μ' = tangent of angle of friction between fill material and the sides of the ditch or trench.

ρ = projection ratio.

ρ_t = steel ratio for temperature and shrinkage reinforcement.

Σ = summation sign -- sum of.

Σo = perimeter of tensile reinforcement.

ϕ = angle of internal friction of a soil.

Additional symbols and notations used in reinforced concrete design discussions conform to those given in the "Reinforced Concrete Design Handbook" of the American Concrete Institute.

SECTION 6

Structural Design - General

1. General Discussion. Structural design is a combination of practical engineering sense and theory. The design process involves two broad phases of the total job. In the first phase the general features or layout of the job are decided upon, tentative selections of the various types of materials to be used are made, and decisions as to the functional requirements of the job are reached. The second phase involves the proportioning of the various parts of the structure to carry the required loads. These two phases are closely related by several factors, one of which is the necessity to produce the required structure economically.

A good structural designer will have a thorough knowledge of construction methods and practice, and he will constantly strive to prepare plans that can be executed without unusual complications; he will weigh the extra costs involved in the construction of complicated details against the functional advantages thereof.

He must ask the following questions over and over during the design process: First, how does this structure and its parts bend, deflect, or move? Second, how can it fail? These are powerful questions. Seldom can the first question be answered with quantitative precision, but it can usually be answered qualitatively. It may be difficult to estimate the magnitude of the strains, rotations, and displacements, but it is imperative that the sense and location of these movements be well defined. Consideration of the second question should include the possibility of failure by collapse of one or all of the structural elements or by sliding, settlement, or some other form of displacement of the entire structure.

Durability is an extremely important factor in the design of hydraulic structures. The durability of a structure is affected by design assumptions, materials, construction practice, and climate. It is not enough to design a structure so that it has sufficient structural strength only; it must be designed to resist structural or physical deterioration for the anticipated life of the structure. Reinforced concrete hydraulic structures that will be subjected to numerous freezing and thawing cycles must be so designed and built that cracks of sufficient size to permit concentrated seepage of water through the structure will be prevented.

Often there are several ways to solve a design problem, two or maybe three of which are definitely superior to the others. Any engineer that has worked in the design field for very long will have assembled a set of design notes, aids, and ways of doing things that he likes and with which he is familiar. His methods may be excellent or they may be inefficient and yet produce sound results. It is far beyond the scope of this handbook to attempt to include even several of the different methods or procedures that might be used to solve any one of the various problems treated. In general, the procedure will be to present one system or method of solution that is practical and adequate and then illustrate it with examples. In some cases various methods will be presented to demonstrate the advantage of one or to facilitate the combination of a set of computations with subsequent steps in the solution of a general problem.

2. Loads. One of the most difficult jobs confronting the structural designer is the determination of the loads to be carried by the structure. Many questions need to be answered. What possible loads can come to the structure during its lifetime including the construction period? In what possible combinations can these various loads act? What are the relationships between magnitude and frequency of the various loads and what effect should these relationships have on design load and unit stress assumptions? What hazards are involved should the structure fail?

Answers to the above questions will provide guidance in selecting design loads. Specially designed structures of unusual magnitude, cost, or complexity, or those in which the hazards are high, should failure occur, will justify a considerable expenditure of time and energy for accurate determination of loads. Such structures are designed for a specific location and purpose and extensive studies in soil mechanics and other fields may be justified.

On smaller structures where the hazards of failure and costs are both comparatively low, it is reasonable and practical to develop standard detailed construction plans that can be used from job to job over a wide range of conditions. The problems of load determination for the design of such structures are somewhat different than for a specific structure to be built at a given location. In the design of standard structures an attempt must be made to determine the average load for each of the various possible loading conditions and then check to be certain that the maximum loads do not encroach on the factor of safety too far. This is more easily said than done because of the many variables such as differences in construction materials at different locations, differences in types of soil, both in foundations and backfill, and differences in climate and exposure conditions.

2.1 Dead Loads. Dead loads are fixed in magnitude, point of application, and direction and act on the structure continuously. They result from the weight of the structure itself and attached appurtenances, and always act in a vertical direction since they result from the pull of gravity.

Extensive tables of weights and specific gravity are given in several handbooks, and it is considered unnecessary to reproduce them here. One such table can be found in the Manual of the American Institute of Steel Construction entitled "Steel Construction"; another is in King's "Handbook of Hydraulics."

The selection of specific weights requires thought and good judgment. Often materials thought of as having fixed specific weights (unit weight) vary considerably in density. For example, the specific gravity of aluminum will vary from about 2.55 to 2.75. The weight of earth in pounds per cubic foot will vary from 65 to 130 depending upon the moisture content, void ratio, and specific gravity of the solid particles. Very well compacted earth may vary from 90 to 130 lbs. per cu. ft.

For standards and small structures, average values must be chosen with good judgment. On specific structures more precise values should be used.

Special caution is justified in those cases where the effective weight of a material is lowered by its submersion in water. Remember that a body heavier than water is reduced in effective weight by an amount equal to the weight of the displaced water, and a body lighter than water by unit weight (specific weight) will displace its weight of water if it floats; if it is submerged, it must be held down by a force equal to the weight of the displaced water minus the weight of the body in air. A more thorough treatment of this subject is found in the section on "Hydraulics - General."

2.2 Live Loads. All loads other than dead loads are live loads. Live loads may be steady or unsteady, fixed, movable or moving; they may vary in magnitude and be applied slowly or suddenly. Fortunately most live loads encountered in the design of hydraulic structures are fixed and relatively steady. However, load fluctuations may be severe and very rapid in a pipe line when a valve is closed suddenly or when there is an abrupt change in flow conditions in the pipe. Bridges are subject to moving loads and impact effects. Earth pressures will vary in intensity but are slow in changing.

2.2.1 Hydrostatic Pressure. Triangular and trapezoidal pressure diagrams that indicate a linear relationship between intensity of pressure and depth are used to represent both water pressure and earth pressure. See Section 5, Hydraulics - General, for a more complete treatment of hydrostatic pressure, or refer to King's "Handbook of Hydraulics", or to any standard textbook on hydraulics.

The solution of a simple but common problem involving hydrostatic pressure is given below. In this illustration the solution is found by three equivalent but slightly different procedures. Note the ease with which this particular problem is solved by the use of drawing ES-4. It will usually be found that this procedure is easiest and most rapid for rectangular areas, the most common case encountered.

Problem: Find magnitude and point of application of resultant pressure on a vertical slice (12 inches wide) of the submerged sluice gate shown below. The gate opening is 5'-0" high. Neglect loads around edge of gate that are directly against the gate frame.

Solution No. 1 - (See Fig. 6.2-1.) Compute intensities of pressure at top and bottom of gate as shown on the drawing. Next compute moment of load diagram about bottom of gate.

	Load	x	Arm	=	Moment
187 x 5.00 =	935 lbs.	x	2.50 ft.	=	2338 ft. lbs.
1/2 x 312 x 5.00 =	780 "	x	5/3 "	=	1300 " "
P =	1715 lbs.		M	=	3638 ft. lbs.

For the resultant P to have the same moment about the base as the load diagram it must be located a distance, e, above the bottom of the gate opening = M ÷ P = 3638 ÷ 1715 = 2.12 ft.

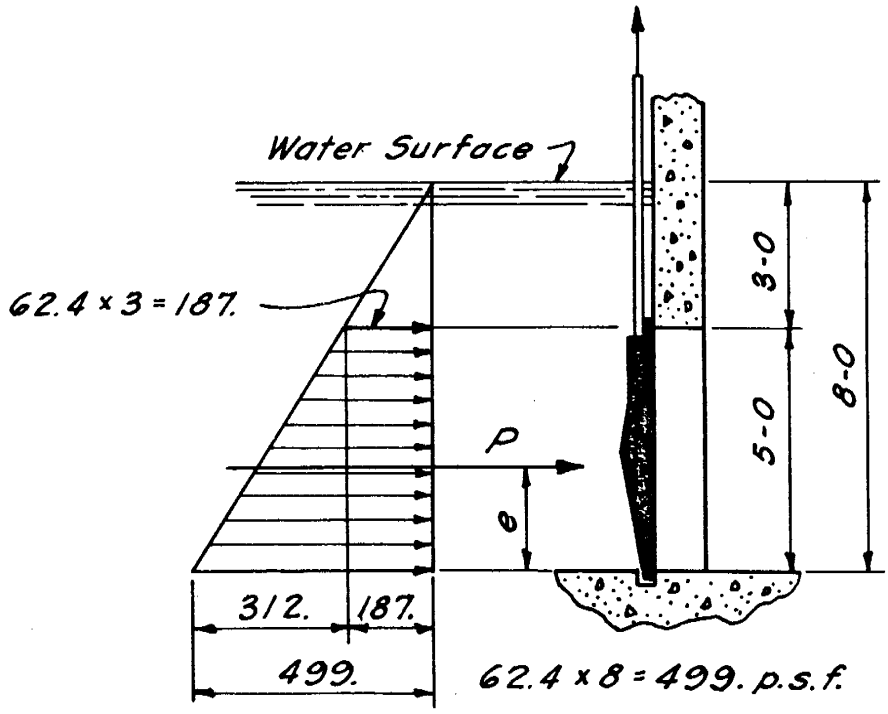


FIG. 6.2-1

Solution No. 2. King's "Handbook of Hydraulics", page 20, 3rd edition:

$$P = wAy = 62.4 \times 5 \times 1 \times 5.50 = \underline{1715} \text{ lbs.}$$

$$x = y + \frac{k^2}{y} = 5.5 + \frac{\frac{5^2}{12}}{5.5} = 5.88 \text{ ft.}; \quad 8.00 - 5.88 = \underline{2.12} = e$$

Solution No. 3. Use drawing ES-4 for y = 8-0, h = 3-0, and w = 62.4

$$S = P = 27.5 \times 62.4 = 1716 \text{ lbs.}$$

$$M = 58.3 \times 62.4 = 3638 \text{ ft.lbs.}$$

$$\frac{M}{P} = \frac{3638}{1716} = \underline{2.12} \text{ ft.} = e$$

2.2.2 Active Lateral Earth Pressure. The determination of precise values for lateral earth pressure is difficult, if not impossible, because of the wide variation in soil types and characteristics, soil moisture relationships, rigidity of the restraining wall, and other factors. The classic theories give reasonably good results in those cases where the physical conditions closely approximate the assumptions on which the theory is based; they may be in serious error for cohesive or wet soils or unyielding walls.

During recent years this subject has received considerable attention as a part of the rapid development of the field of Soil Mechanics. As yet, however, there has been no reasonably easy and accurate solution to this general problem. Extensive tests of the physical and structural properties of soil are needed to make use of the currently available advanced methods of solution. Only in unusual cases would the type and size of structures built by the Soil Conservation Service justify such tests.

Hence, it has been arbitrarily decided to use the classic method of Coulomb and the other customary practice based on equivalent fluid pressures as the two most applicable methods for our work.

Coulomb's method is applicable only to dry, noncohesive, permeable soils of high structural strength best exemplified by relatively clean sand or gravel.

Coulomb's general equation for active earth load on a yielding wall is:

$$P = 1/2\ wH^2 \frac{\sin^2(\theta - \phi)}{\sin^2\theta \sin(\theta + Z)\left(1 + \sqrt{\frac{\sin(Z + \phi)\sin(\phi - i)}{\sin(\theta + Z)\sin(\theta - i)}}\right)^2} \qquad (6.2\text{-}1)$$

where

P = total pressure per linear foot of wall in lbs.
w = specific (unit) weight of soil in lbs. per cu. ft.
H = height of wall in ft.
θ = angle between back face of wall and horizontal.
ϕ = angle of internal friction of the soil.
Z = friction angle between soil and back of wall.
i = angle of surcharge between fill slope and horizontal.

Coulomb's equations are based on the assumption of linear variation in pressure; hence, P always acts on the surface under consideration at H/3 above the base.

Dry, clean sand and gravel evidence values of the variables involved approximately as indicated below:

Variable	Units	Min.	Max.	Remarks
w	lbs/ft^3	100	120	Increases with compaction
ϕ	degrees	35	45	Increases with compaction
Z	degrees	20	25	Sand against concrete

Note: If the fill will be subjected to vibration from any source, the value of Z should be taken as zero. Tractors used for backfilling will usually provide sufficient vibration to warrant reducing Z to zero in addition to producing a surcharge on the wall.

This nomenclature is illustrated in fig. 6.2-2 below.

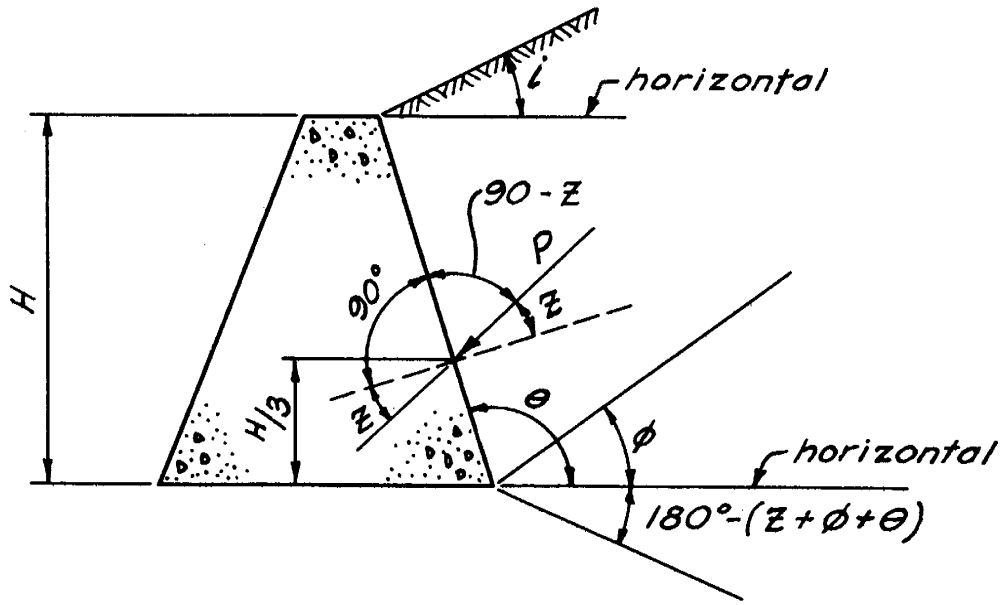


FIG. 6.2-2

Culmann has developed a graphical solution to Coulomb's equation that has practical value, particularly for those cases in which the backfill surface is irregular or carries a surcharge. This graphical solution is illustrated by the following examples.

Example 1 - (See fig. 6.2-3)

Step 1. Draw a sectional elevation of the wall and the earth fill surface to scale, compute the values of angles θ and i, and then tabulate the values of θ, i, ϕ, Z, and w.

Step 2. Lay out lines AB and AC as shown in fig. 6.2-3. Line AB is laid off from A, the heel of the wall, above the horizontal line AD so that the angle BAD = ϕ. Line AC is laid off from A below the horizontal line AD so that the angle DAC = $180^{\circ} - (Z + \phi + \theta)$.

Step 3. Compute the weight of the various earth fill slices indicated. An easy way to do this in this case is: Extend the earth fill line past point M and draw the line AM through A so that the angle NMA is 90 degrees. Scale or compute the length of the line AM. Next lay off distances 8, 10, 12, etc., feet from point 0 along the earth fill line to the scale used in drawing the wall. Draw the lines A8, A10, A12, etc., thus defining various earth slices whose weights are to be computed. In a case such as this, the use of an even increment of distance along the earth fill slopes will facilitate the computations as will be seen later.

The weight of the earth slice (1 foot thick) 0-8-A-0 is determined by multiplying the base of the triangle = 8 feet by the height = 12.02 feet by one-half by the unit weight of the earth. (See the computations on fig. 6.2-3.) The weight of the slice 0-10-A-0 is found by adding the weight of the slice increment 8-A-10-8 to the weight of the slice 0-8-A-0. The weight of the slice increment is computed as shown in fig. 6.2-3. Add increments to obtain weight of other slices.

Step 4. To some convenient scale lay off the weights of the various earth fill slices along the line AB with the zero point of the scale at A, thus locating points W8, W10, etc.

Step 5. Next draw lines from W8, W10, etc., parallel to the line AC to an intersection with the corresponding lines A8, A10, etc.; i.e., the line from W8 parallel to AC is drawn to an intersection with the line A8, etc.

Step 6. Connect the points of intersection obtained in step 5 with a smooth curve and draw a tangent to this curve parallel to the line AB.

Step 7. From the point of tangency found in step 6 draw a line paralle to AC back to the line AB. The length of line at the scale used in plotting the weights along AB is the value of the resultant load on the wall = P. Th horizontal and vertical components of P can be found graphically as indicated in fig. 6.2-3.

Step 8. Locate the point of application and direction of the resultant load P. The point of application of the load is on the wall a vertical distance of one-third H above its base. The direction of P is defined by the angle Z.

This graphical solution can be checked by substitution in Coulomb's general equation as follows:

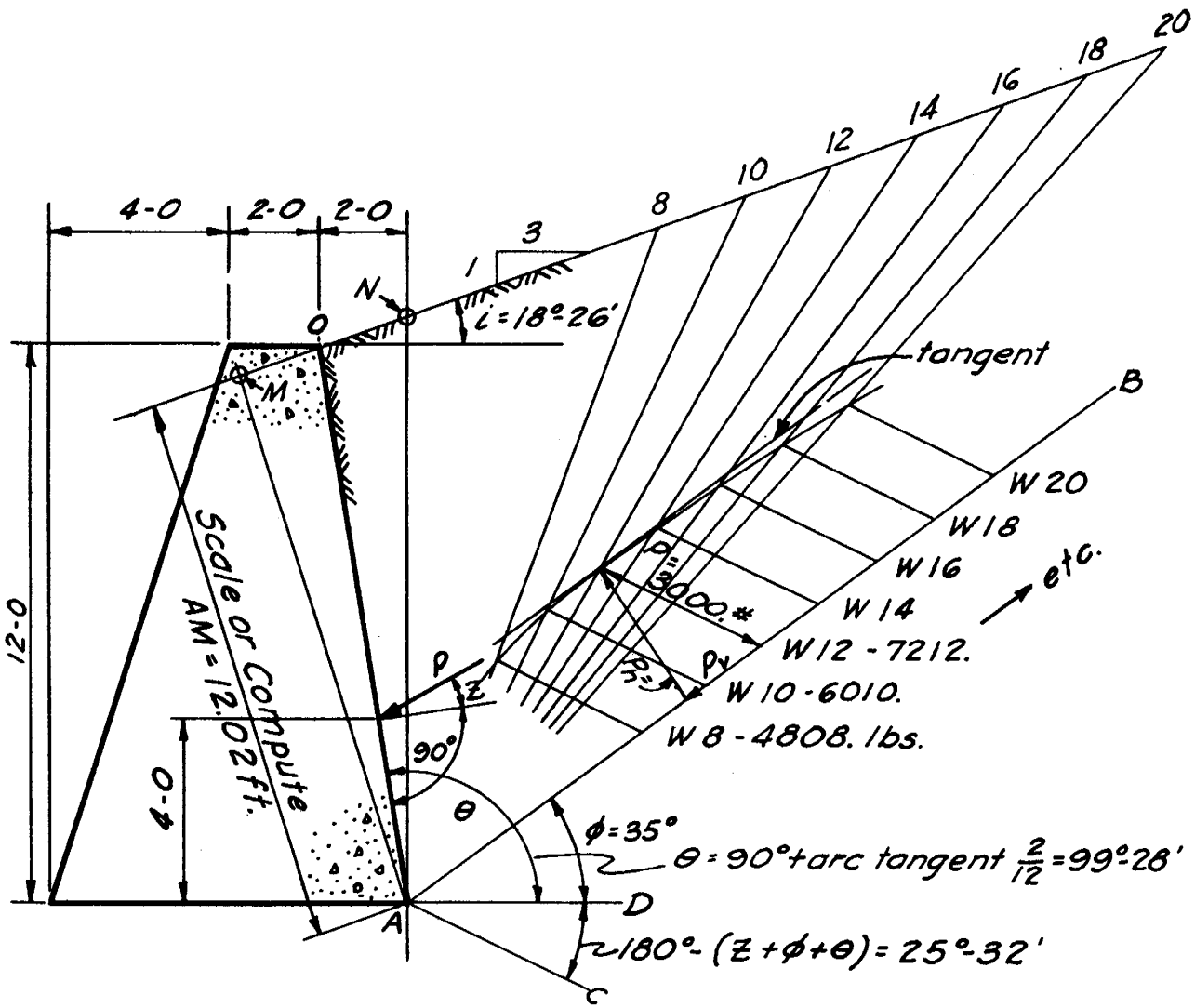
$$P = \frac{0.5 \times 100 \times 12^2 \times \sin^2(99^\circ 28' - 35^\circ)}{\sin^2 99^\circ 28' \sin(99^\circ 28' + 20^\circ)\left(1 + \sqrt{\dfrac{\sin(20^\circ + 35^\circ)\sin(35^\circ - 18^\circ 26')}{\sin(99^\circ 28' + 20^\circ)\sin(99^\circ 28' - 18^\circ 26')}}\right)^2}$$

$$= \frac{7200 \times (0.90234)^2}{(0.98638)^2(0.87064)\left(1 + \sqrt{\dfrac{0.81915 \times 0.28513}{0.87064 \times 0.98778}}\right)^2} = 3000 \text{ lbs.}$$

The horizontal and vertical components of P can be computed from the following equations:

$$P_h = P \cos\left[Z + (\theta - 90^\circ)\right] = 3000 \cos 29^\circ 28' = 2620 \text{ lbs.}$$

$$P_v = P \sin\left[Z + (\theta - 90^\circ)\right] = 3000 \sin 29^\circ 28' = 1470 \text{ lbs.}$$



$w = 100$ lbs/ft^3
$\theta = 99^\circ\text{-}28'$
$\phi = 35^\circ$
$Z = 20^\circ$
$i = \text{arc tan } \frac{1}{3} = 18^\circ\text{-}26'$

Compute AM

$$AN = 12 + \frac{2}{3} = \frac{38}{3}$$

$$\frac{AM}{AN} = \frac{3}{\sqrt{10}} \text{ or } AM = \frac{38}{3} \times \frac{3}{\sqrt{10}}$$

$$AM = 12.02$$

Compute Slice Weights

$$W8 = \frac{1}{2} \times 8 \times 12.02 \times 100 = 4808 \text{ lbs.}$$

$\Delta_2 W$ = weight of 2 ft. slice increment

$$= \frac{1}{2} \times 2 \times 12.02 \times 100 = 1202$$

$$W10 = W8 + \Delta_2 W = 4808 + 1202 = 6010 \text{ lbs.}$$

$$W12 = 6010 + 1202 = 7212 \text{ lbs.}$$

Etc.

FIG. 6.2-3

If the angles i and Z equal zero and angle θ = 90 degrees, then Coulomb's general equation reduces to

$$P = 1/2\ wH^2 \frac{1 - \sin \phi}{1 + \sin \phi} \qquad (6.2\text{-}2)$$

which is identical to Rankin's equation for this case.

This case is illustrated in the following example. Its solution is probably most easily accomplished by the above equation (6.2-2) unless a scale drawing of the wall and other tools are readily available for the graphical construction. The graphical solution is demonstrated to point out some important relationships for this case and to further illustrate its use.

<u>Example 2</u> - (See fig. 6.2-4)

A two-foot surcharge has been assumed in this case; this is approximately equivalent to a heavy crawler-type tractor that might be expected to operate close to the top of the wall during construction operations. Paragraph 3.2.18, page 137, of the "Standard Specifications for Highway Bridges" of the American Association of State Highway Officials requires the placement of a two-foot surcharge of earth where highway traffic can be expected within a distance of one-half H from the top of the wall. In either case, the two-foot surcharge is a logical addition to the load on the wall. In such cases Z should be taken equal to zero.

The graphical solution was made following the procedure outlined in example 1. Where the top of the fill is horizontal, as in this case, the bisector of the angle TAB defines the wedge of maximum thrust so that the maximum value of P can be drawn without constructing the curve.

The value of P = 2290 lbs, found from the graphical construction, is the load on the surface OA. The value of the load on AT can be found most easily by converting P into an equivalent fluid pressure diagram as shown in fig. 6.2-4. From this equivalent fluid pressure diagram the load on the wall AT becomes a trapezoidal pressure diagram. The resultant of the trapezoidal pressure diagram can be found as indicated in fig. 6.2-4, or the total load and moment about the base can be found very easily by the use of drawing ES-4 as follows: For h = 2, y = 13, and w = 27.1,

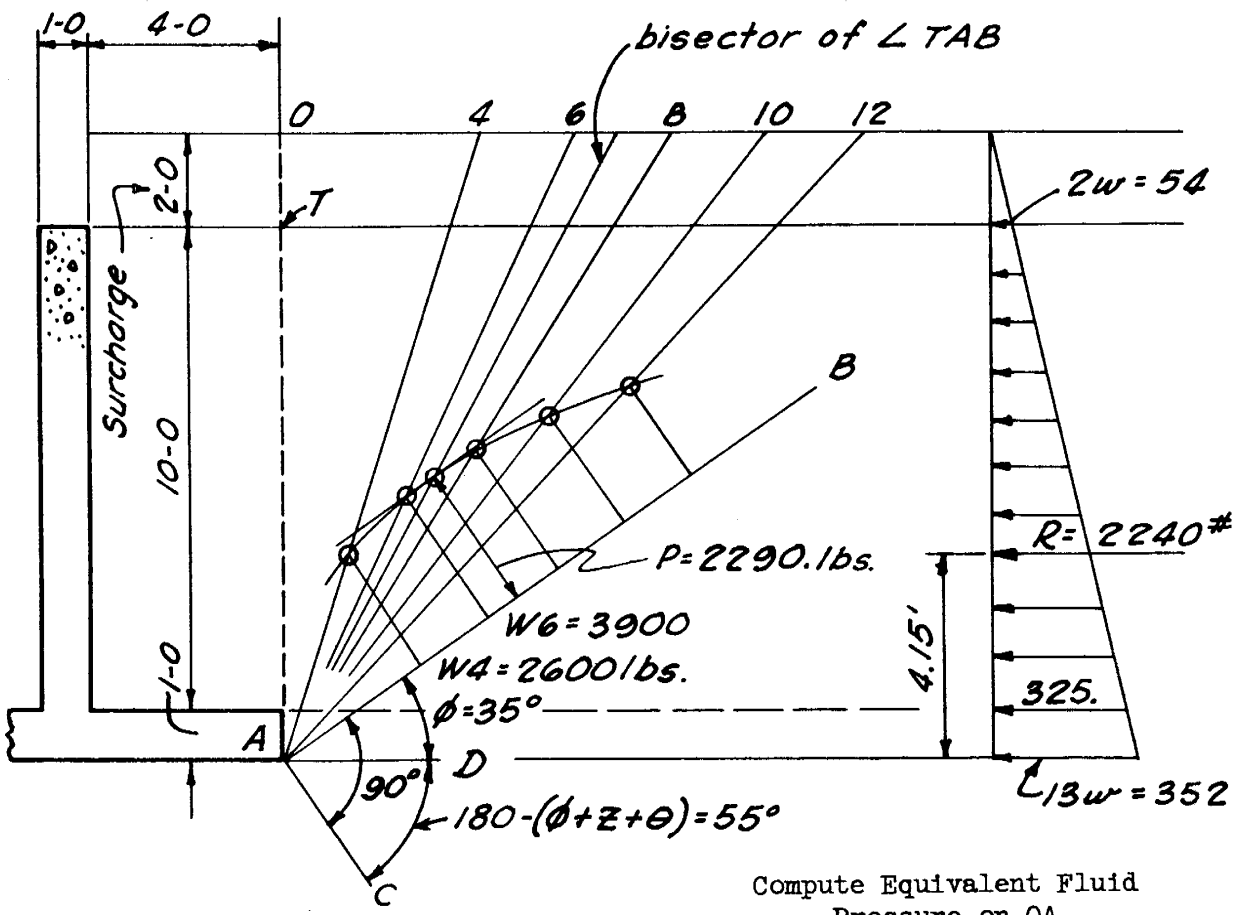
$$R = S = 82.5 \times 27.1 = 2240 \text{ lbs.}$$

$$M = 342.8 \times 27.1 = 9290 \text{ ft.lbs.}$$

The solution for P given in fig. 6.2-4 may be checked by equation (6.2-2).

$$P = 1/2\ wH^2 \frac{1 - \sin \phi}{1 + \sin \phi} = 1/2 \times 100 \times 13^2 \frac{1 - \sin 35^\circ}{1 + \sin 35^\circ}$$

$$= 8450 \times \frac{0.42642}{1.57358} = 2290 \text{ lbs.}$$



$\theta = 90^{\circ}$, $Z = 0$

$w = 100$ lb/ft^3

Compute Slice Weights

$W4 = 1/2 \times 4 \times 13 \times 100 = 2600$ lbs

$\Delta_2 W$ = weight of 2 ft slice increment

$= 1/2 \times 2 \times 13 \times 100 = 1300$ lbs

$W6 = 2600 + 1300 = 3900$ lbs

Etc.

Compute Equivalent Fluid Pressure on OA

$1/2\ wH^2 = 2290$ lbs

$$w = \frac{2290 \times 2}{13^2} = 27.1 \text{ lb/ft}^3$$

Resultant load and moment from dwg. ES-4 for y = 13 and h = 2

$R = 82.5 \times 27.1 = 2240$ lbs

$M = 342.8 \times 27.1 = 9290$ ft lbs

$$\frac{M}{R} = \frac{9290}{2240} = 4.15 \text{ ft}$$

FIG. 6.2-4

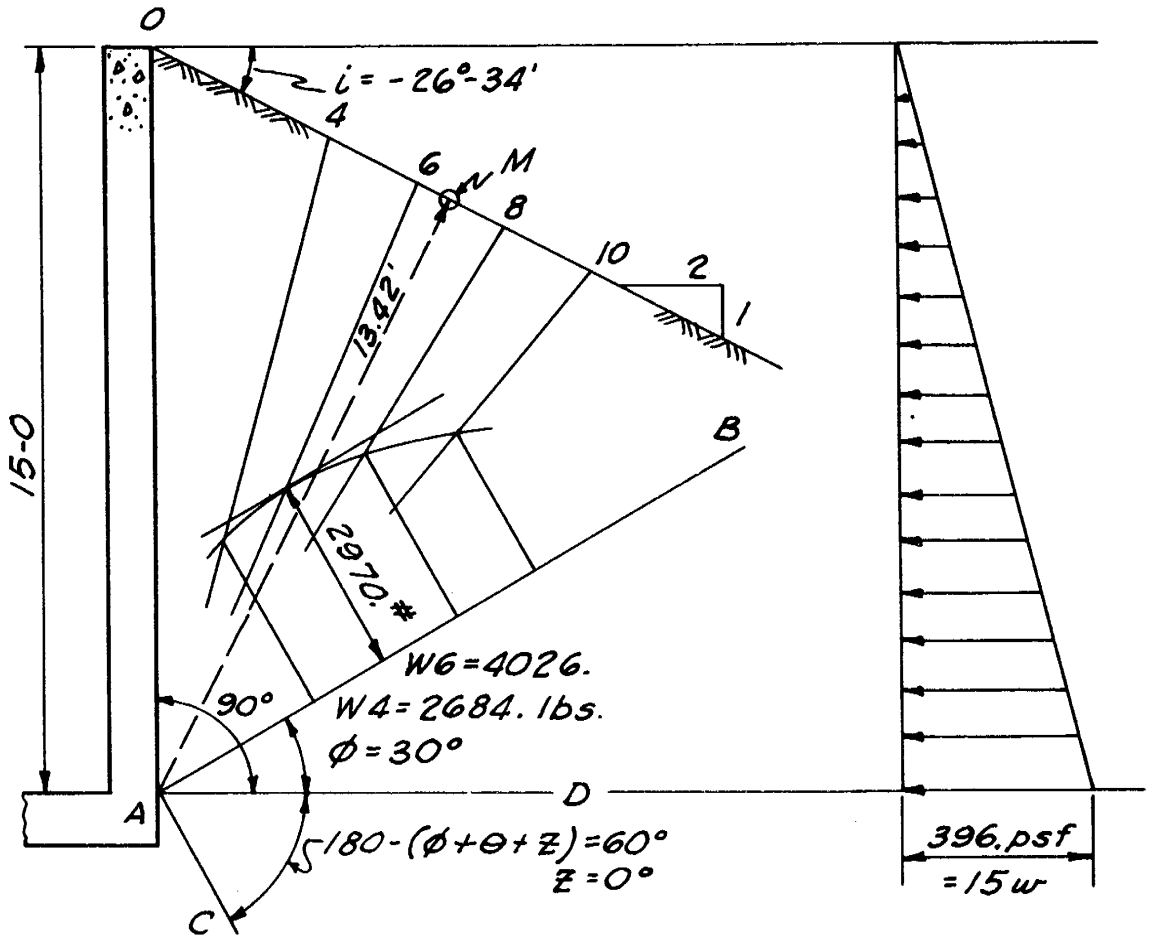
If the angle $\theta = 90$ degrees and angle Z = zero, then Coulomb's equation reduces to

$$P = 1/2\ wH^2 \frac{\cos^2 \phi}{\left(1 + \sqrt{\frac{\sin \phi \sin(\phi - i)}{\sin(\theta - i)}}\right)^2} \qquad (6.2\text{-}3)$$

Example 3 - (See fig. 6.2-5)

In this example a negative value of the angle i has been assumed and an illustration solved graphically and algebraically. The graphical solution is straightforward following the rules previously given. Signs of the angles must be watched in the algebraic solution. For example, in this case, $\sin(\phi - i) = \sin 30^\circ - (-26^\circ 34') = \sin(30^\circ + 26^\circ 34')$.

Note that when Z = 0, the direction of the resultant load is normal to the wall; then if the wall is vertical (θ = 90 degrees), the resultant load P is horizontal.



AM = 15 cos 26°34' = 13.42 ft.

W4 = 1/2 x 4 x 13.42 x 100 = 2684 lbs

$\Delta_2 W$ = 1/2 x 2 x 13.42 x 100 = 1342 lbs

W6 = W4 + $\Delta_2 W$ = 2684 + 1342 = 4026

W8 = W6 + $\Delta_2 W$; etc.

Equivalent fluid pressure

$1/2\ wH^2 = 2970$

$w = \frac{2970 \times 2}{225} = 26.4\ \text{lbs/ft}^3$

FIG. 6.2-5

Algebraic Solution: (Based on equation 6.2-3.)

$$P = 1/2\ wH^2 \frac{\cos^2 \phi}{\left(1 + \sqrt{\frac{\sin\phi \sin(\phi - i)}{\sin(\theta - i)}}\right)^2} = \frac{0.5 \times 100 \times 225 \times \cos^2 30^\circ}{\left(1 + \sqrt{\frac{\sin 30^\circ \sin(30^\circ + 26^\circ 34')}{\sin(90^\circ + 26^\circ 34')}}\right)^2}$$

$$= \frac{11250 \times 0.866^2}{\left(1 + \sqrt{\frac{0.50 \times 0.835}{0.894}}\right)^2} = 2970 \text{ lbs.}$$

As pointed out previously, the lateral earth pressure depends on numerous characteristics of the backfill.

The shearing strength of the soil is determined by tests that give values of the angle of internal friction, ϕ, and of cohesion. For relatively clean sand and gravel the angle of internal friction is high. It usually varies between 35 and 45 degrees and is affected very little by moisture content; the cohesion is negligible. The moisture content of clay affects its shearing strength so that the angle ϕ may vary from zero for fully saturated clay to about 30 degrees for dry clay; cohesion is usually quite high in dry clay with considerable decrease with increased moisture.

Permeability of the backfill may vary from very high values for clean sands and gravels to practically zero for consolidated clays.

The density of the backfill obviously is a factor in the lateral pressures that it will exert.

The most important single characteristic for most soils other than clean sands or gravels is the moisture content. Numerous tests have shown that earth pressures go up with an increase in moisture content of the backfill. The amount of moisture in a backfill depends upon several factors which can be grouped under two principal categories. They are: (1) factors affecting the availability of free water; and (2) those factors affecting the disposal of excess free water.

In general, there are two sources of surface supply and one of underground supply. Water may reach the backfill of a wall by direct rainfall, overland flow, or lateral underground seepage.

Good surface drainage and a layer of well-graded, relatively impervious soil of high density, that shrinks little on drying, placed on top of the backfill will help to reduce infiltration of water from the surface.

In open permeable soils above a low water table (below the bottom of the wall by several feet), water that does penetrate the soil surface seeps vertically through the profile and causes little if any increase in lateral pressure. Even in permeable soils, if the water table is high and no drainage is provided, the amount of water that infiltrates through

the surface may soon fill all the voids in the soil thus completely saturating it with an accompanying large increase in lateral pressure due to the addition of the hydrostatic load.

In soils of low permeability, adequate drainage is difficult and costly. The drainage usually provided for retaining walls and comparable structures is entirely inadequate for clay soils with the result that if a clay backfill is allowed to become wet, as it usually will, there is a gradual increase in lateral pressure with the increase in moisture until at complete saturation the backfill acts very much like a liquid.

Freezing temperatures and the subsequent formation of ice layers in fine-grained soils constitute another possible source of increased lateral earth pressures.

In view of the many factors that affect lateral earth pressures, it is somewhat presumptuous to present the following table; however, we are continually faced with the design of various types of structural elements that must satisfactorily resist lateral earth pressures. The previous discussion and the following table 6.2-1 should constitute a reasonable guide for estimating probable lateral earth pressures for small structures.

TABLE 6.2-1

Equivalent Fluid Pressures for Compacted Backfills of Various Soils with no Surcharge

Type of Soil	Permeability	Shearing Strength	Approx. Dry Weight lbs/ft^3	Equivalent Fluid Weight in lbs per cu ft		
				Ordinary Drainage		Saturated
				A[1]	B[2]	
1. Clean sand, gravel or sand-gravel mixture	high	high	110	30	35	80
2. Well-graded sand, silt, and clay mixture	low	high	125	40	65	85
3. Sandy, clayey silt	low	good	100	45	80	90
4. Clay	very low	low	100	90	90	120
5. Fluid mud, sluiced silty clay or clay		none	- -		--	135

[1]Condition A - Infiltration from rainfall is only source of surface water; good surface drainage away from the wall; low water table several feet below bottom of wall.

[2]Condition B - Subject to rainfall and inflow of surface runoff from adjacent areas; poor surface drainage; high water table near or above bottom of wall.

Drainage of the backfill is usually essential if pressures are to be kept low enough to permit economical designs. However, the cost of providing adequate drainage may be so high for clay and other fine-grained soils that it becomes economical to design relatively low walls for high lateral pressures and use only nominal customary drainage. The availability of pit-run sand or gravel for use as drains and the cost thereof will affect this decision. Obviously, it is impossible to state any fixed rule--the decision should be based on comparative cost studies for any particular structure and site.

A minimum amount of drainage should be provided for all walls subject to lateral earth pressure. Weep holes of 3 or 4-inch diameter with the lowest practical free outlet should be provided at 5 to 10 feet spacing with a coarse sand or gravel filter and collector running the full length of the wall. Such a filter should be at least 18 inches thick and have a horizontal dimension of 18 inches up, depending upon the soil type. In clayey silts and clays, the horizontal dimension of the filter might well be made equal to half the height of the wall if filter material is readily available and cheap.

It is often advantageous to use a perforated collector pipe in the filter and outlet this pipe through the wall at intervals of 50 to 100 feet. Usually such a pipe need not exceed four inches in diameter if it is placed on a one-percent grade and does not extend more than 50 feet on each side of the outlet. Such pipe should be durable and of sufficient strength to withstand the loads imposed on it.

Very good drainage of clay backfills can be accomplished by placing a large filter of clean, coarse sand or gravel or sand-gravel mixture back of the wall as indicated in fig. 6.2-6. This recommendation is based on work reported by Prof. Gregory P. Tschebotarioff of Princeton University in Paper No. 2374, Vol. 114, 1949, of the Transactions of the American Society of Civil Engineers entitled "Large-scale Model Earth Pressure Tests on Flexible Bulkheads." The following quotation is from page 431 of the above reference:

> "The use of a sand dike sloping away from the bulkhead of a natural (1:1.7) slope was found to be fully effective in reducing the fluid lateral pressures transmitted to the bulkhead from the unconsolidated fluid clay backfill behind the dike. The pressures against the bulkhead were no greater than those exerted by a backfill composed entirely of sand.
>
> "The interposition between the fluid clay backfill and the bulkhead of a vertical sand blanket with a width equal to the bulkhead height was found to be just as effective as the interposition of a sand dike. When the width of the blanket was equal to one-half the bulkhead height, it was only approximately one-half as effective, and it was completely ineffective when its width was equal to one-tenth of the bulkhead height. In the latter case the lateral pressures transmitted to the bulkhead from the unconsolidated clay backfill were no smaller than those of a fluid.
>
> "The foregoing statements should be taken only as indications of order of dimension, accurate to approximately $\pm$ 10 percent."

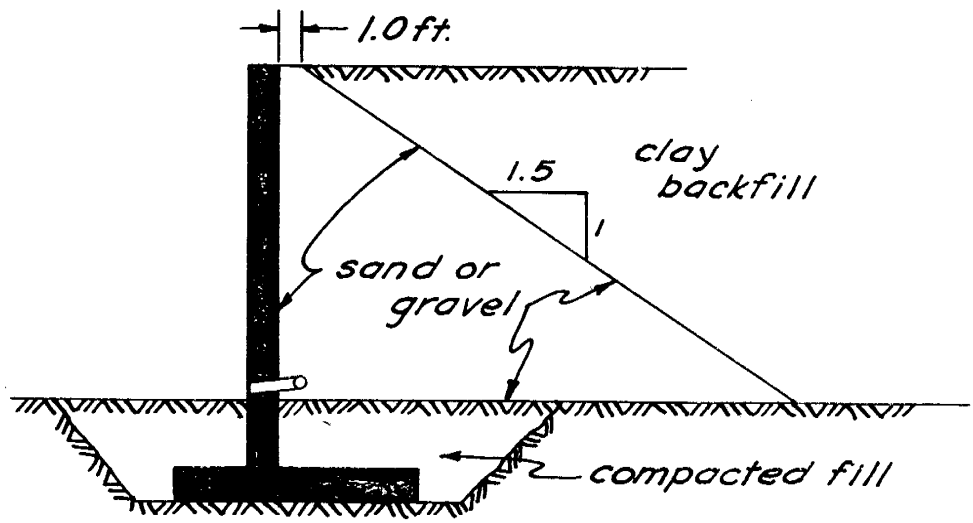


FIG. 6.2-6

Soils that contain appreciable amounts of clay are subject to excessive shrinkage. During dry periods, extensive cracking of such soils can be expected; cracks are quite apt to open up at the plane of contact between a wall and the backfill. Such cracks may extend to depths of 10 feet or more. Where such walls and backfills are subject to overland flow, the minimum lateral pressure for which the wall should be designed is full hydrostatic pressure, since water will enter the crack between the wall and the backfill and develop full hydrostatic pressure for the depth of the crack before the soil can swell to its original volume and slow down the infiltration of water at the surface of the ground.

The following references are good sources of information on lateral earth pressures:

1. "Soil Mechanics in Engineering Practice" by Karl Terzaghi and Ralph B. Peck; published by John Wiley and Sons.
2. "Fundamentals of Soil Mechanics" by Donald W. Taylor; published by John Wiley and Sons.
3. "The Theory and Practice of Reinforced Concrete" by Clarence W. Dunham; published by McGraw-Hill Book Co.

2.2.3 Passive Lateral Earth Pressure. Passive lateral earth pressures are more highly indeterminate than active pressures. As would be expected, less research has been done on them than on active pressures and the methods of evaluating passive pressure are more directly dependent upon theoretical mechanics without experimental verification. Hence, it is wise to be conservative in the use and evaluation of passive resistance in the design of structures.

Passive resistance of the earth in front of a retaining wall or cutoff wall under a dam is sometimes used to increase the factor of safety of the structure against sliding. Such practice is not recommended, but may be necessary in some cases.

Coulomb's equation for passive pressure is:

$$P_p = \frac{wH^2}{2}\left(\frac{\csc\theta\,\sin(\theta+\phi)}{\sqrt{\sin(\theta-Z)} - \sqrt{\frac{\sin(\phi+Z)\sin(\phi+i)}{\sin(\theta-i)}}}\right)^2 \qquad (6.2\text{-}4)$$

It is conservative and customary to ignore wall friction in computing passive pressure. Then when $Z = 0$ and the wall is vertical ($\theta = 90^\circ$), equation (6.2-4) reduces to:

$$P_p = \frac{wH^2}{2}\,\frac{\cos^2\phi}{\left(1 - \sqrt{\frac{\sin\phi\,\sin(\phi+i)}{\cos i}}\right)^2} \qquad (6.2\text{-}5)$$

When $Z = 0$, $\theta = 90^\circ$, and the earth surface is level ($i = 0$), equation (6.2-4) becomes:

$$P_p = \frac{wH^2}{2}\,\frac{1+\sin\phi}{1-\sin\phi} \qquad (6.2\text{-}6)$$

Table 6.2-2 gives approximate values of the ratio of passive lateral pressure to vertical pressure for a range of soil types and conditions. These values will give reasonable values of passive resistance and are especially useful where the earth surface is confined by the apron of a dam or where it carries a surcharge.

TABLE 6.2-2

Type of Soil	Ratio of Passive Lateral Pressure to Vertical Pressure	
	Dry	Saturated
1. Clean sand, gravel, or sand-gravel mixture	4	3
2. Well-graded sand, silt, and clay mixture	3	2
3. Sandy clayey silt	2	1
4. Clay	2	0

Unless the structure rests on a permeable foundation in which the water table is several feet below the bottom of the wall or cut-off wall, the saturated condition should be assumed in design.

Probable frost depths also influence the selection of values to be used. In silty clays, and clays especially, thaw and the accompanying change from ice to water will soften the soil and reduce its resistance to lateral movement to almost nothing. Unless the soil is very well drained, the passive resistance should be taken as zero for the normal average frost depth.

2.2.4 Loads on Underground Conduits. This subject is especially important to engineers of the Soil Conservation Service because of the numerous field conditions encountered that require such load determinations. Drop inlet barrels, inverted siphons, tile drains in unusually deep cuts, and culverts are examples of structures that are subjected to the loads discussed below.

Much of the following material has been taken from a paper by Prof. M. G. Spangler of Iowa State College entitled "Underground Conduits - An Appraisal of Modern Research" which was published as Paper No. 2337 of the 1948 Transactions of the American Society of Civil Engineers, Vol. No. 113. In the following discussion wherever direct quotations are made, they are from the above paper unless specifically designated otherwise. Prof. Spangler's equations and figures have been re-numbered to conform to the system herein.

"Underground conduits, in general, may be divided into two main classes on the basis of construction conditions under which they are installed, that is (1) ditch conduits and (2) projecting conduits.

(1) 'Ditch conduits' are structures installed and completely buried in narrow ditches in relatively passive and undisturbed soil. Examples of this class of conduits are sewers, drains, and water mains (see fig. 6.2-7).

(2) 'Projecting conduits' are structures installed in shallow bedding with the top of the conduit projecting above the surface of the natural ground and then covered with an embankment, as shown in fig. 6.2-7. Railway and highway culverts are good illustrations of this class of conduits. Conduits installed in ditches wider than about two or three times their maximum horizontal breadth may also be treated as projecting conduits.

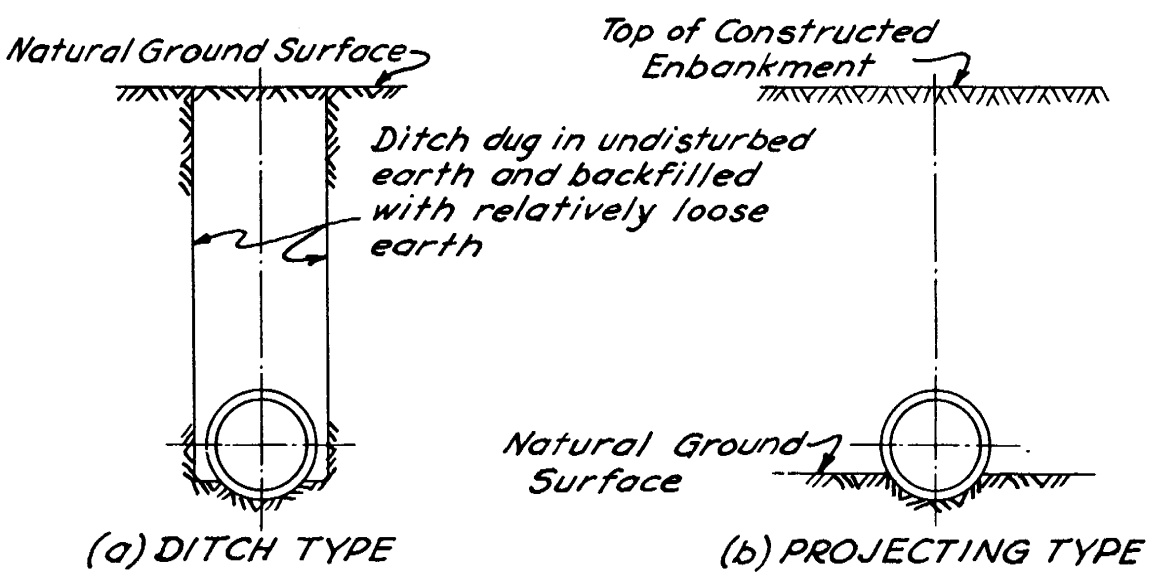


FIG. 6.2-7

Essential Elements of Typical Conduits

Ditch Conduits

"When a conduit is placed in a ditch not wider than about two or three times its outside breadth and covered with earth, the backfill will tend to settle downward. This downward movement or tendency for movement of the soil in the ditch above the pipe produces vertical frictional forces or shearing stresses along the sides of the ditch which act upward on the prism of soil within the ditch and help to support the backfill material. Assuming the cohesion between the backfill material and the sides of the ditch to be negligible, the magnitude of these vertical shearing stresses is equal to the active lateral pressure exerted by the earth backfill against the sides of the ditch multiplied by the tangent of the angle of friction between the two materials. This assumption of negligible cohesion is justified because: (a) Even when the ditch is dug in and backfilled with cohesive material, considerable time must elapse before effective cohesion between the backfill material and the sides of the ditch can develop after backfilling; and (b) the assumption of no cohesion yields the maximum probable load on the conduit. The maximum load may develop at any time during the life of the conduit due to heavy rainfall or other causes which may eliminate or greatly reduce cohesion between the backfill and the sides of the ditch."

The total vertical pressure within the ditch at the elevation of top of conduit is given by the following equation:

$$P = \gamma b_d^2 \frac{1 - e^{-\alpha H_c}}{2 K \mu'} \qquad (6.2\text{-}7)$$

where

P = total vertical pressure in the width b_d at the top of the conduit in pounds per linear foot of conduit.

γ = the unit weight of the backfill in pounds per cubic foot.

b_d = breadth or width of the ditch or trench at the top of the conduit.

e = 2.7183 = base of Naperian logarithms.

$$\alpha = \frac{2 K \mu'}{b_d} \qquad (6.2\text{-}8)$$

$$K = \frac{\sqrt{\mu^2+1} - \mu}{\sqrt{\mu^2+1} + \mu} \qquad (6.2\text{-}9)$$

= ratio of active lateral pressure to vertical pressure (from Rankine's theory).

μ' = tangent of the angle of sliding friction between backfill and adjacent earth.

μ = tangent of the angle of internal friction of the backfill material.

"The proportion of this total pressure that will be carried by the conduit will depend upon the relative rigidity of the conduit and of the fill material between the sides of the conduit and the sides of the ditch. In the case of very rigid pipes such as burned clay, concrete, or heavy cast-iron pipe, the side fills may be relatively compressible and the pipe itself will carry practically all the load, P. On the other hand, if the pipe is a relatively flexible, thin-walled pipe and the side fills are thoroughly tamped in at the sides of the pipe, the stiffness of the side fills may approach that of the conduit and the load on the structure will be reduced by the amount of load the side fills are capable of carrying.

"For the case of rigid ditch conduits with relatively compressible side fills, the load will be:

$$W_c = C_d \gamma b_d^2 \qquad (6.2\text{-}10)$$

"For the case of flexible pipes and thoroughly compacted side fills having the same degree of stiffness as the pipes, the load will be:

$$W_c = C_d \gamma b_d b_c \qquad (6.2\text{-}11)$$

in which W_c is the total load on the conduit, and

$$C_d = \frac{1 - e^{-\alpha H_c}}{2 K \mu'} \qquad (6.2\text{-}12)$$

The solution of equation (6.2-12) is facilitated by the use of the curves shown on drawing ES-15 which give values of C_d for various typical kinds of backfill material.

"The width of ditch, b_d , is the actual width of a normal, parallel-sided ditch. In case the ditch is constructed with sloping sides or the conduit is placed in a subditch at the bottom of a wider trench, experiments have shown that the width of ditch at or slightly below the top of the pipe is the proper width to use when determining the load.

"These ditch conduit formulas (equations 6.2-10 and 6.2-11) with proper selection of the physical factors involved give the maximum loads to which any particular conduit may be subjected in service. On the other hand, because of the development of cohesion, any particular conduit may escape the maximum load for a long time, sometimes until its removal for other causes than load failure. Experiments and field observations show that the load on a conduit at the time the fill is completed is usually less than it will be at some later time. This condition accounts for the fact that sewers and other conduits which have been observed to be structurally sound immediately upon completion are sometimes found to be cracked some months or years later."

A sample computation of C_d follows: Assume $K\mu' = 0.13$, and $H_c + b_d = 10$, then

$$C_d = \frac{1 - e^{-2K\mu' \frac{H_c}{b_d}}}{2K\mu'} = \frac{1 - e^{-2 \times 0.13 \times 10}}{2 \times 0.13}$$

$$= \frac{1 - \frac{1}{13.46}}{0.26} = 3.56$$

This value checks the value obtained from the curve on drawing ES-15.

Projecting Conduits

Drop inlet barrels and culverts, as usually built, are examples of projecting conduits. A thorough treatment of loads on projecting conduits requires no less than the amount of discussion devoted to it by Prof. Spangler in the reference given previously. A thorough study of this technical paper is recommended. Since space does not permit reproduction of this part of Prof. Spangler's discussion, only the actual working procedure necessary for the solution of our ordinary problems will be given below.

Almost all of the projecting conduits encountered in the work of the Soil Conservation Service will fall inside the range of variables indicated below.

The settlement deflection ratio, δ, will vary between zero (0) and one (1). Values recommended for use are given in drawing ES-22.

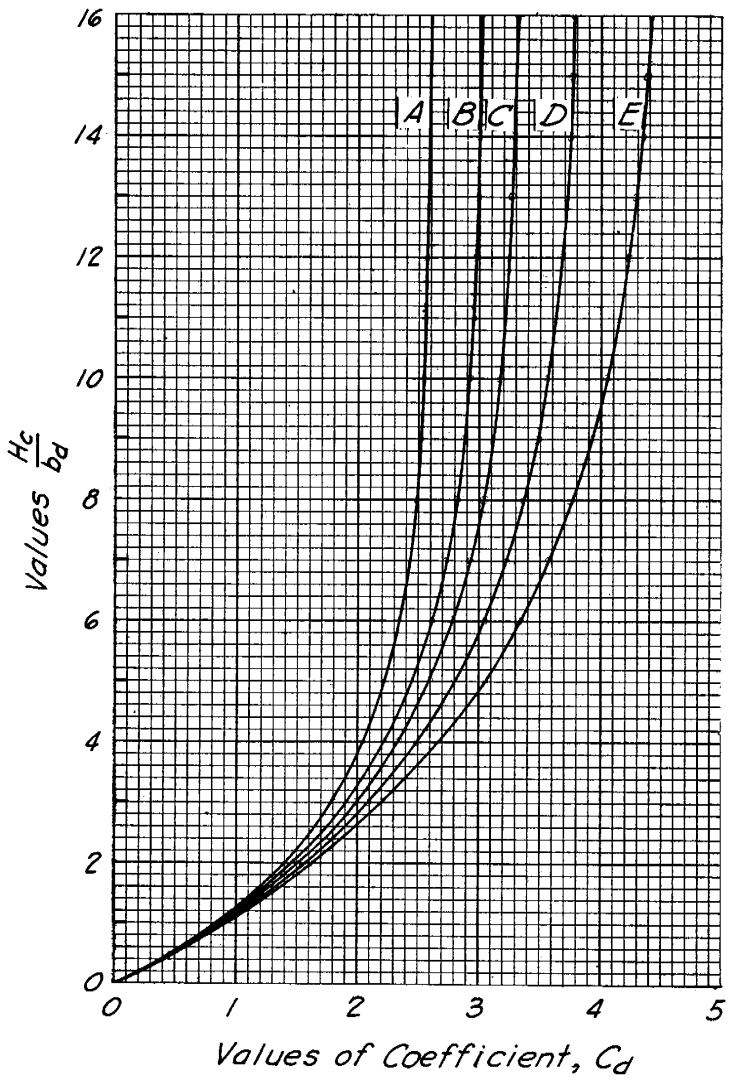
The projection ratio, ρ, is equal to the distance between the natural ground surface and the top of the conduit divided by the width of the conduit, b_c. For economically proportioned conduits this value will rarely exceed 1.6 and may vary from zero to 1.6.

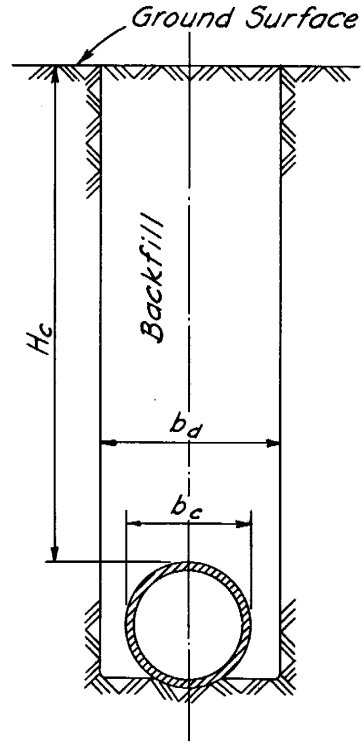
Hence, the product $\delta\rho$ may vary between zero and 1.6, the limits indicated on the curve on drawing ES-22.

Values of $K\mu$ will, in almost all cases, lie between 0.11 and 0.192, the maximum possible value. Fig. 6.2-8 shows a plotting of $K\mu$ as the ordinate against μ as the abscissa with corresponding values of ϕ = arc tan μ.

It is possible for values of $K\mu$ to be below 0.11, but not probable. Only in case of a fully saturated soil with high clay content would such values be possible and these circumstances are unusual. Values of μ (and hence $K\mu$) can be determined from soil shear tests, and this should be done on important work. If there is doubt as to the proper value of $K\mu$ for use in any specific case, use the highest probable value. It is obvious from the curves in drawing ES-22 that the vertical load on a conduit

STRUCTURAL DESIGN: LOADS ON DITCH CONDUITS





Ditch Conduit
Typical Installation

Curve	Conditions of Applicability	$K\mu = K\mu'$	γ
A	Minimum for granular materials	0.1924	100.
B	Maximum for sand and gravel	0.165	110.
C	Maximum for saturated top soil	0.150	110.
D	Ordinary maximum for clay	0.130	120.
E	Maximum for saturated clay	0.110	135.

W_c = load on conduit in lbs. per linear ft.
H_c = height of back fill above top of conduit in ft.
b_c = breadth of conduit in ft.
b_d = breadth of ditch at top of conduit in ft.
γ = unit weight of backfill in lbs. per cu. ft.

For Rigid Conduits

$$W_c = C_d \gamma b_d^2$$

For Flexible Pipes with Compacted Side Fills

$$W_c = C_d \gamma b_d b_c$$

Example: Find load per ft. on standard strength, 24 in. diam. R/c culv. pipe laid in 42 in. trench under 22 ft. of ordinary clay backfill.

$b_c = \frac{24+(2\times3)}{12} = 2.50$; $\gamma = 120.$

$b_d = 42 \div 12 = 3.50$; $H_c = 22.0$

$H_c/b_d = 22 \div 3.5 = 6.28$ then from curves $C_d = 3.1$

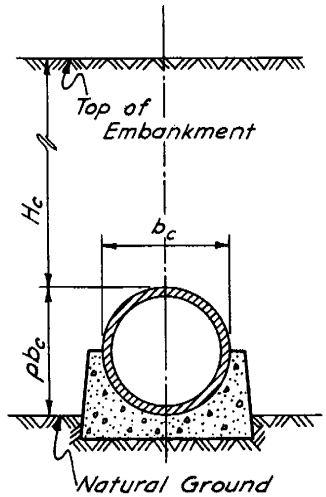
$W_c = C_d \times \gamma \times b_d^2$
$= 3.1 \times 120 \times 3.5^2 = 4560$ lbs.

REFERENCE Trans. Amer. Soc. of Civil Engrs., Vol. 113, pg. 316 - "Underground Conduits - An Appraisal of Modern Research" by Prof. M.G. Spangler.

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING STANDARDS UNIT

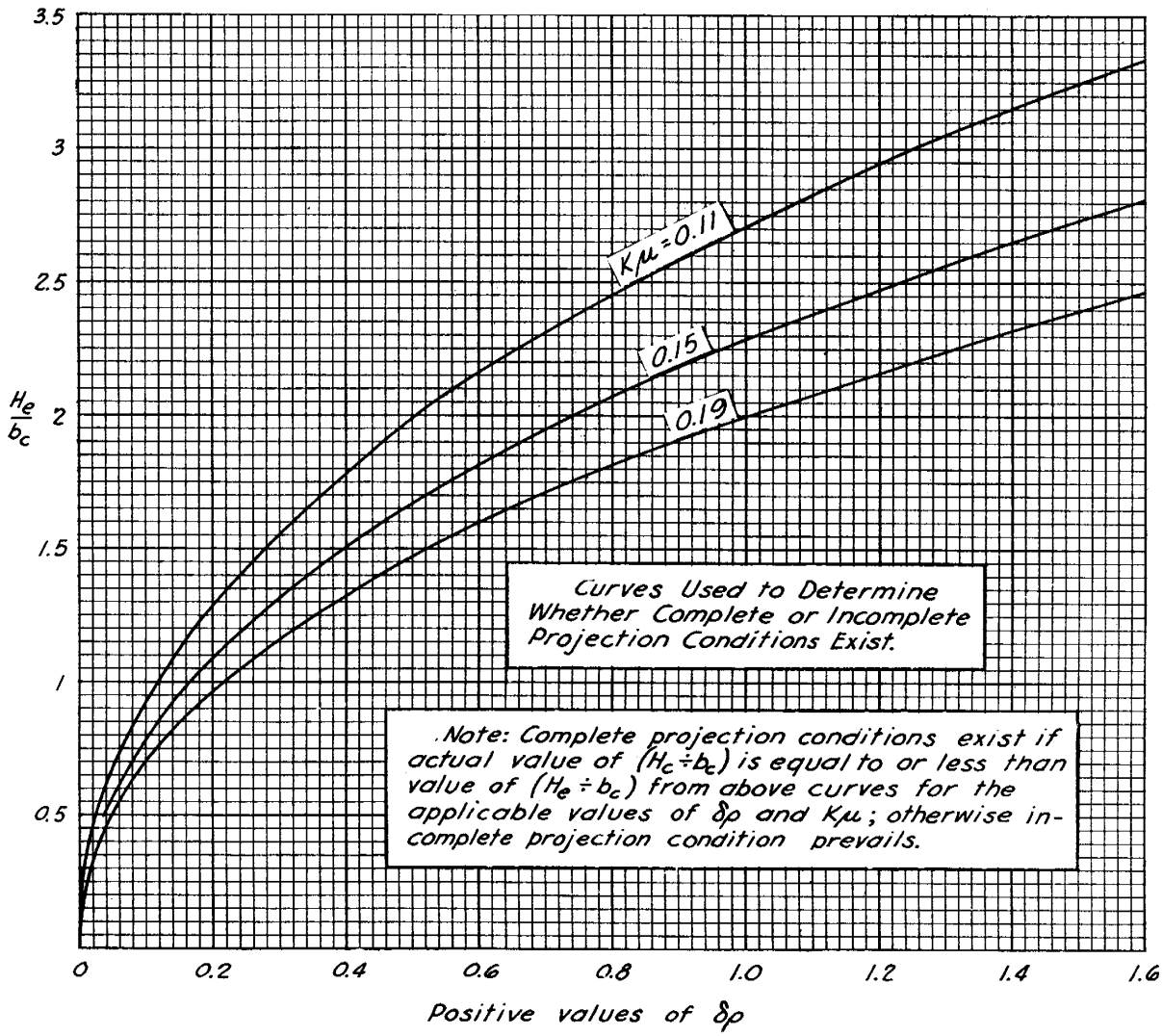
STANDARD DWG. NO. ES-15
SHEET 1 OF 1
DATE 3-31-50

STRUCTURAL DESIGN:
LOADS ON RIGID PROJECTING CONDUITS



Foundation Material	δ
Rock or other unyielding material	1.00
Dense mixture of sand and gravel	0.80
Glacial till, dense, well graded	0.70
Clay, dense, consolidated, firm	0.70
Silt, loose sand or other yielding soils	0.50
Very soft, loose, wet, yielding material	0.30

Embankment Material	$K\mu$
Sand, gravel, or well graded sand, silt, clay mixture	0.19
Sandy, clayey silt or dry clay	0.17
Clay, permanently wet	0.15
Clay or silty clay, permanently saturated	0.13



REFERENCE Trans. ASCE, vol. 113 "Underground Conduits - An Appraisal of Modern Research" by M. G. Spangler, M. ASCE.

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

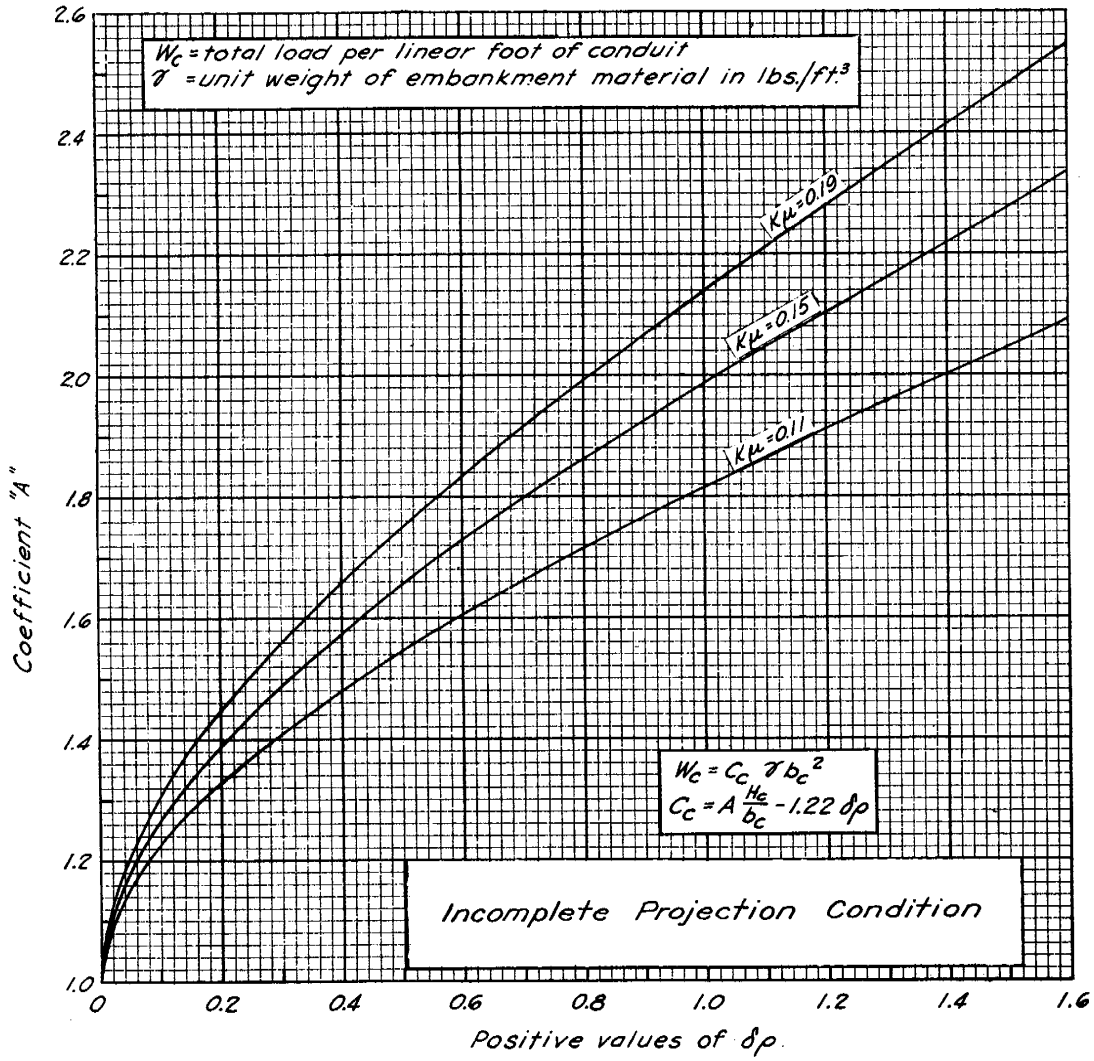
ENGINEERING STANDARDS UNIT

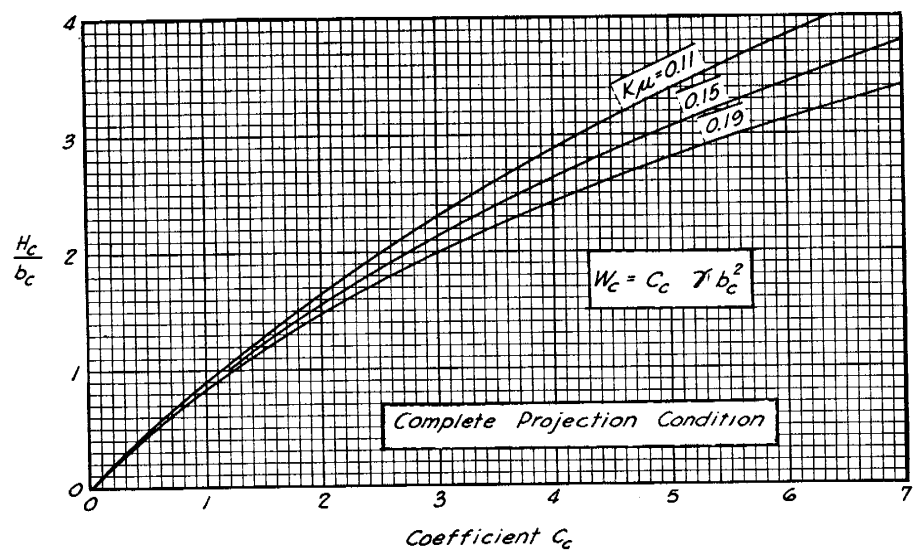
STANDARD DWG. NO. ES-22

SHEET 1 OF 2

DATE 10-19-50

STRUCTURAL DESIGN:
LOADS ON RIGID PROJECTING CONDUITS

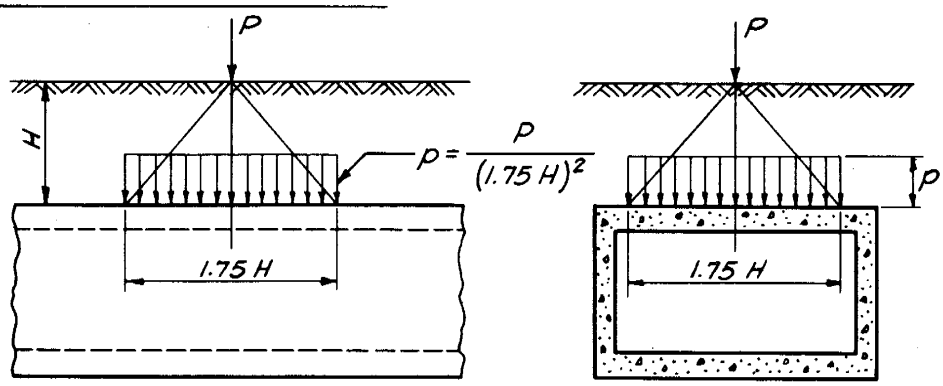




REFERENCE Trans. ASCE, vol. 113 "Underground Conduits - An Appraisal of Modern Research" by M. G. Spangler, M. ASCE.	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING STANDARDS UNIT	STANDARD DWG. NO. ES-22 SHEET 2 OF 2 DATE 10-19-50

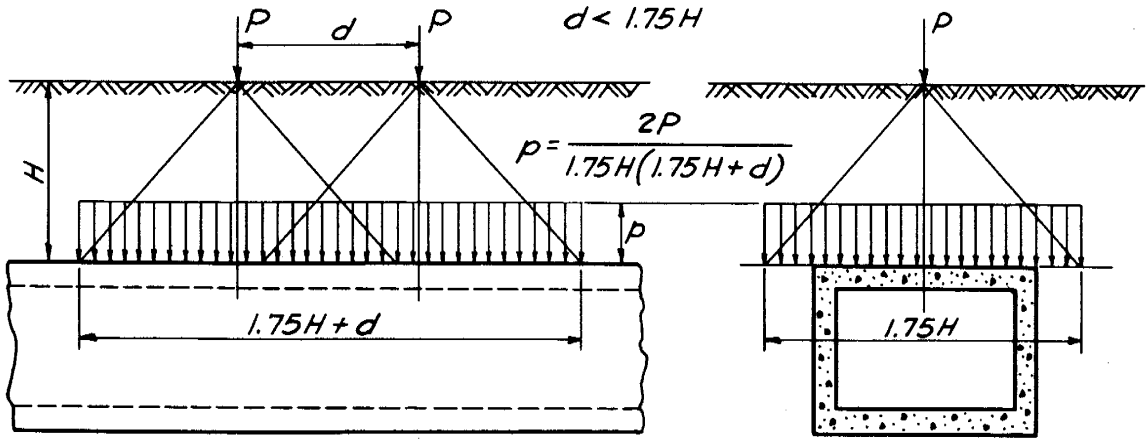
STRUCTURAL DESIGN: DISTRIBUTION OF SURFACE LOADS THROUGH EARTH FILL.

Single Concentrated Load



Elevation — Cross-section

Two Concentrated Loads - (overlapping areas of influence)



Elevation — Cross-section

Notes - General

1. When $H \leq 2$ ft. loads shall be treated as concentrated loads applied directly to the conduit.
2. Where $H > 10$ ft. wheel loads may be neglected.
3. When $H > 3$ ft. neglect effect of impact from moving wheel loads.
4. When $H \leq 3$ ft. add 20 percent to wheel loads for impact effect.
5. In unusual cases determine pressures from Boussinesq equations or other more precise methods. — See references listed below.

References:

1. "Fundamentals of Soil Mechanics" by D.W. Taylor - pages 250-266.
2. "The Theory and Practice of Reinforced Concrete" by C.W. Dunham, p.207.
3. "Underground Conduits - An Appraisal of Modern Research" by M.G. Spangler in Trans. A.S.C.E. Vol. 113 for 1948.
4. "Standard Specifications for Highway Bridges" of The American Association of State Highway Officials - fifth edition 1949 - page 143.

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING STANDARDS UNIT	STANDARD DWG. NO. ES - 25 SHEET 1 OF 1 DATE 4 - 26 - 50

increases with an increase in $K\mu$. Hence, to be on the conservative side, the highest probable value of $K\mu$ that might exist during the life of the structure should be used in computing design loads that will produce maximum moments and shears in the top and bottom slabs of rectangular conduits. (To determine maximum shears and moments in the side walls of rectangular conduits, the vertical and horizontal loads should be determined by use of the lowest probable value of $K\mu$.) For circular conduits, if in doubt as to the proper value, use $K\mu = 0.19$. A table of approximate values of $K\mu$ for different soils is shown on drawing ES-22.

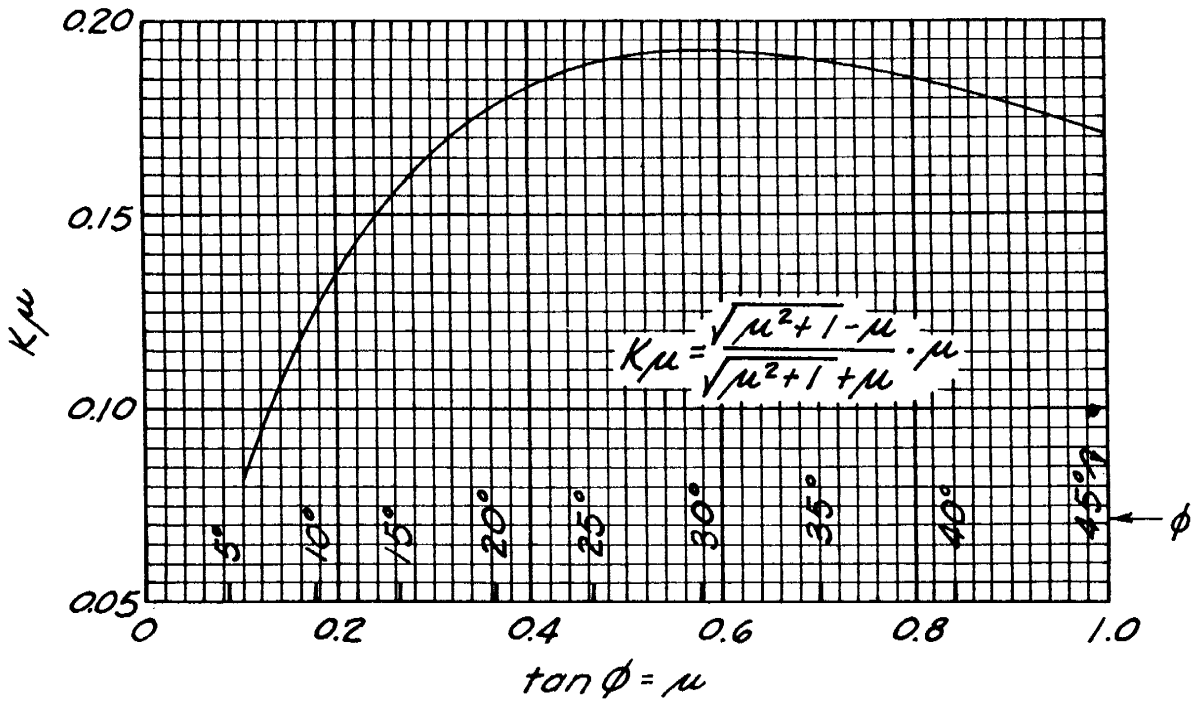


FIG. 6.2-8

The method of computing C_c (see drawing ES-22) used herein deviates slightly from the correct equations given by Prof. Spangler. A small error in results is due to an approximation which is compatible with the inherent lack of precision in estimating design values of δ and $K\mu$.

Computation procedure and the use of drawing ES-22 are illustrated by examples given below.

Example 1

Problem: It is proposed to build an earth dam with a culvert-type spillway as a retarding structure for upstream flood control. Preliminary design indicates that the height of fill above the top of the conduit, H_c, will be 38 feet and that a twin 5x5 ft monolithic reinforced concrete conduit is required to provide the necessary discharge capacity.

Preliminary structural design of the conduit indicates that the dividing wall between the two 5x5 conduits will need to be 8 in. wide, that the sidewalls will each have a width of 9 in. at the top of the conduit, and that the top and bottom slabs of the conduit will have thicknesses of about 15 and 16 in. respectively. The foundation for the conduit is a glacial till, dense and well graded and the embankment material will be a dense (compacted) well-graded mixture of sand, silt, and clay with a unit weight, γ, of 132 lbs per cu ft. Since it is necessary to excavate overburden to sound foundation for both the earth embankment and the conduit, the conduit will project above the excavated grade for its full height. Find the maximum load on top of the conduit per foot of conduit length and the average load intensity in lbs per sq ft.

Step 1. Select the proper value of δ from the table on drawing ES-22, compute the projection ratio, ρ, and then compute the product $\delta\rho$.

From ES-22, for a foundation material of dense, well-graded glacial till, $\delta = 0.70$.

The projection ratio, ρ, equals the difference in elevation between the top of the conduit and the ground line divided by the top width of the conduit, b_c. In this case the above difference in elevation is equal to the height of the conduit = $[(15 + 16) \div 12] + 5 = 7.58$ ft, and the top width of the conduit, b_c, = $[(9 + 9 + 8) \div 12] + (2 \times 5) = 12.17$ ft. Then $\rho = 7.58 \div 12.17 = 0.623$, and $\delta\rho = 0.70 \times 0.623 = 0.436$.

Step 2. Select a proper value of $K\mu$ from drawing ES-22 for the type of embankment material to be used. The highest probable value of $K\mu$ for a compacted, well-graded mixture of sand, silt, and clay is 0.19.

Step 3. Compute the value of $(H_c \div b_c)$ from data on H_c given in the problem and the value of b_c computed in step 1. $(H_c \div b_c) = (38 \div 12.17) = 3.12$.

Step 4. From the "Curves Used to Determine Whether Complete or Incomplete Projection Conditions Exist" on sheet 1 of 2 of ES-22, find the proper value of $(H_e \div b_c)$ for the values of $\delta\rho$ and $K\mu$ determined above and compare this value with the value of $(H_c \div b_c)$ found in step 3. If $(H_c \div b_c)$ is greater than $(H_e \div b_c)$, the incomplete projection condition exists; and if $(H_c \div b_c)$ is equal to or less than $(H_e \div b_c)$, the complete projection condition exists.

In our present example for $\delta\rho = 0.436$ and $K\mu = 0.19$, $(H_e \div b_c) = 1.38$. From step 3 $(H_c \div b_c) = 3.12$, which is greater than 1.38, hence an incomplete projection condition exists.

Step 5. From the proper curve on sheet 2 of ES-22, compute the efficient C_c.

In this example, since the incomplete projection condition exists, the value of A from the curves at the top of sheet 2 of ES-22 and

compute C_c from the following equation:

$$C_c = A\ (H_c \div b_c) - 1.22\,\delta\rho \qquad (6.2\text{-}13)$$

If the complete projection condition exists, find the value of C_c directly from the curves at the bottom of sheet 2 of ES-22.

For the example at hand, find A = 1.69; then $C_c = (1.69 \times 3.12) - (1.22 \times 0.436) = 4.74$.

Step 6. Compute W_c = load per linear foot of conduit from the following equation:

$$W_c = C_c\ \gamma\ b_c^2 \qquad (6.2\text{-}14)$$

In this case $W_c = 4.74 \times 132 \times 12.17^2 = 4.74 \times 1.32 \times 10^2 \times 1.48 \times 10^2 = 9.27 \times 10^4 = 92{,}700$ lbs.

Next compute the load in lbs per sq ft on the conduit = $W_c \div b_c = (9.27 \times 10^4) \div 12.17 = 0.762 \times 10^4 = 7620$ lbs per sq ft.

It is significant that the ratio of the actual load to the weight of earth above the top of the conduit = $7620 \div (132 \times 38) = 1.52$.

Note: If the final structural design deviates appreciably from the dimensions used in the load determination, revision is necessary.

Example 2

Problem: Find the load intensity in lbs per sq ft on top of the conduit of example 1 when H_c = 10 ft. Assume that conduit dimensions, foundation conditions, and embankment are the same as in example 1.

Step 1. From example 1, b_c = 12.17 ft and $\delta\rho = 0.436$.

Step 2. From example 1, $K\mu = 0.19$.

Step 3. $H_c \div b_c = 10 \div 12.17 = 0.823$.

Step 4. From example 1, $H_e \div b_c = 1.38$ for $K\mu = 0.19$ and $\delta\rho = 0.436$. Since $(H_c \div b_c) = 0.823$ (from step 3) is less than $(H_e \div b_c) = 1.38$, the complete projection condition exists.

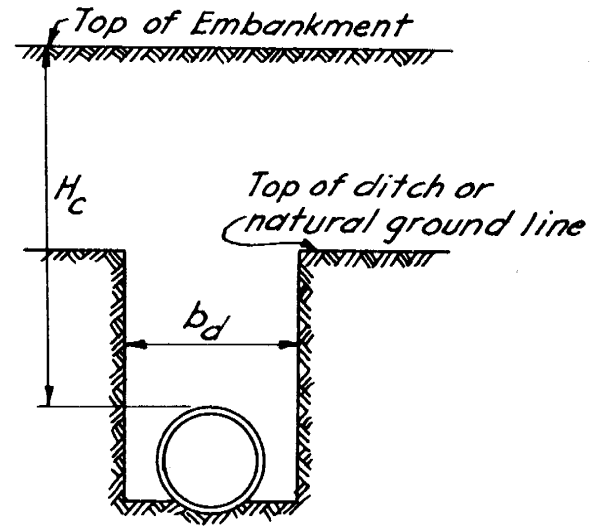
Step 5. Since a complete projection condition exists, find C_c directly from the curves at the bottom of sheet 2 of ES-22. For $H_c \div b_c = 0.823$ and $K\mu = 0.19$, $C_c = 0.97$.

Step 6. $W_c = C_c\ \gamma\ b_c^2 = 0.97 \times 132 \times 12.17^2 = 0.97 \times 1.32 \times 10^2 \times 1.48 \times 10^2 = 1.895 \times 10^4 = 18{,}950$ lbs. $W_c \div b_c = 18{,}950 \div 12.17 = 1560$ psf.

The ratio of load to weight of earth above the conduit = $1560 \div (132 \times 10) = 1.18$.

Negative Projecting Conduits

Negative projecting conduits are constructed in a narrow trench as shown in fig. 6.2-9. They differ from ditch conduits in that the embankment extends above the top of the ditch (natural ground line) for a distance which is considerably greater than the distance from the top of the conduit to the top of the ditch.



Typical Negative Projecting Conduit

FIG. 6.2-9

The mathematical theory for this condition has not been published. If the backfill between the top of the conduit and the natural ground line were filled with loosely placed material, the loads on the conduit would probably be less than on a normal projecting conduit with equal fill height. If the backfill in the ditch (as indicated above) were compacted to equal or greater density than the adjacent natural ground, the loads on the conduit would probably approach the loads on a fully projecting conduit.

Since the theory is not available for this load condition, and since dam construction methods do not permit the placement of loose fill around or above the conduit (because of seepage and possible piping failure), it is recommended that conduits installed in the negative projection condition be designed as projection conduits with a positive projection ratio of 0.8.

It is difficult to see how a conduit could be safely installed in an earth dam embankment as a negative projecting conduit, since all of the backfill would have to be thoroughly compacted.

Distribution of Loads. The total earth fill load per longitudinal foot of conduit, W_c, may be assumed to be uniformly distributed in the transverse direction over the width of the conduit, b_c. Hence, the load in lbs per sq ft on the conduit is equal to $W_c \div b_c$. For circular or elliptical conduits a slightly more accurate and conservative assumption is that the load will be uniformly distributed over the width of the conduit subtended by an angle of 120 degrees symmetrical about the vertical axis of the conduit.

Surface Loads. The following method of computing the effect of surface loads on underground conduits is an approximation. More accurate methods based on the Boussinesq equations are available, but their application is somewhat tedious and the added refinement will not compensate for the cost of applying the method on most of our work.

A single concentrated load applied directly to the earth fill surface is assumed to produce a uniform intensity of pressure at a distance H below the fill surface on a square area of influence whose side dimension is 1.75H. When two or more equal concentrated loads are so located on a horizontal fill surface that their areas of influence overlap for a given value of H, the intensity of pressure shall be determined by dividing the total of such loads by the area contained within the outside boundary of their combined areas of influence. These conditions are illustrated in drawing ES-25.

The specifications of the American Association of State Highway Officials, fifth edition, may be used where applicable instead of the above.

2.2.5 Highway Loads. The latest edition (1949) of the "Standard Specifications for Highway Bridges" of the American Association of State Highway Officials provides thoroughly tried and tested load assumptions for highways. These specifications should be accepted as standard and used unless the specifications of the highway agency having jurisdiction over the project being designed require more conservative design assumptions.

2.2.6 Wind Loads. The usual structures designed and built by the Soil Conservation Service are not materially affected by wind; and where wind loads are significant, ordinary methods of load determination are adequate.

Such methods are based on the assumption that the wind direction is parallel to the earth's surface. The intensity of pressure on a vertical surface normal to the wind is given by the following equation:

$$p = KV^2 \qquad (6.2\text{-}15)$$

where

p = pressure on a vertical flat surface normal to wind direction in pounds per square foot (psf).
K = a shape factor coefficient.
V = wind velocity in miles per hour.

Values of K vary over a range from 0.002 to 0.006 depending upon the shape of the body. A design value commonly used is K = 0.003. Assuming a wind velocity of 100 miles per hour and K = 0.003, p equals 30 psf, which is another commonly used design value.

For surfaces that are inclined to the wind direction, the intensity of normal pressure is given with satisfactory accuracy by Duchemin's equation which follows:

$$p_n = p \frac{2 \sin \theta}{1 + \sin^2 \theta} \qquad (6.2\text{-}16)$$

where

p_n = normal unit pressure on inclined surface in psf.
p = pressure on flat vertical surface normal to wind direction in psf from equation (6.2-15).
θ = angle between the horizontal and the inclined surface.

For example: Assume a design value for p = 30 psf. Then the normal pressure on a roof having a rise of 10 feet in a 40-foot span is computed as follows: θ = arc tan 0.5; sin θ = 0.447

$$p_n = 30 \frac{2 \times 0.447}{1 + (0.447)^2} = 22.4 \text{ psf.}$$

An excellent discussion of this general subject is contained in an article entitled "Wind Pressure on Structures" by Mr. George E. Howe, which was published in the March 1940 issue of Civil Engineering, Vol. 10, No. 3. This reference contains an extensive bibliography on this subject.

2.2.7 Snow Loads. Snow loads, including sleet (ice), act vertically and their magnitude is a function of the wind velocity, geographic location of the structure, slope of the loaded surface, and other factors. Snow loads vary from zero to 30 psf or more depending upon the above factors. Where sleet is probable, a design load of 10 psf is often assumed to provide for its effect on the structure.

Since there are relatively few design situations in our work that require consideration of snow and ice loads, you are referred to standard handbooks and other available references for additional data.

2.2.8 Ice Pressures. Ice pressures used in the design of dams have ranged from zero to about 50,000 lbs per lin ft. The problem is highly indeterminate and recommendations vary over a wide range.

Ice pressure depends upon (1) thickness of the ice sheet, (2) rate and range of change in ice temperature which is not directly correlated or equal to air temperature, (3) the temperature gradient through the ice sheet, (4) the degree of confinement of the ice sheet at its boundaries, and (5) other factors. One of the most recent discussions of this subject is found in a paper entitled "Thrust Exerted by Expanding Ice Sheet" by Edwin Rose, Esq. in the Transactions of the American Society of Civil Engineers, Vol. 112, page 871, which contains a bibliography on this subject.

Most structures designed or built by the Soil Conservation Service are not subject to damaging ice pressures for several reasons. If damage or excessive load from freezing ice is anticipated, careful consideration should be given to possible changes in layout and design to avoid such damaging loads. Such changes can often be made with less increase in cost than would be involved in designing to withstand the possible ice pressure. Earth fill berms at normal pool elevation may be designed to hold the water surface away from reinforced concrete drop spillways, drop inlets, chutes, trash racks, and other structural elements that might otherwise be subjected to ice pressure; ice pressures have no significant effect on earth embankments principally because of the ability of the earth to yield without damage and because of the sloping surface of contact between the earth and the ice which limits the normal pressure to that required to overcome frictional resistance.

Mr. Clarence W. Dunham, in his book "The Theory and Practice of Reinforced Concrete", recommends a load of 700 lbs per lin ft of retaining wall applied at the earth surface to provide for frost (ice) pressure where poor surface drainage exists and there is a probability of ice formation in the upper layers of the soil profile. This recommendation emphasizes the necessity for, and advantages of good surface drainage and good internal drainage of the soil profile, conditions under which the above load need not be considered. Please refer to the discussion on the effect of drainage on active lateral earth pressures in part 2.2.2 of this section of the handbook.

3. Design and Analysis

Experience and judgment are the foundation of all structural design which is basically a cut-and-try process. The first trial or preliminary design may be close to the final or far away, depending upon the ability of the designer to estimate (from experience and judgment) the sizes and proportions of the various structural elements. The final step in the design process is to check the design in all of its elements by an analysis of the structure to be certain that it will resist the loads that come to it without exceeding the permissible stresses. The designer's goal is to produce plans for an adequate, safe, and economical structure; analysis is but one step in the process and is important only as a check on the designer's judgment.

3.1 Fundamental Requirements. A structural designer must meet certain basic requirements of experience, knowledge, and ability. His value as a designer will increase as his experience grows; experience is gained both from doing and from reading about what others have done.

The basic principles of structural analysis are relatively few in number and, with a few exceptions, are relatively simple. The structural designer must have a thorough, clear, and complete knowledge and conception of equilibrium; force; moment; couple; the basic laws and propositions regarding the composition and resolution of forces, moments, and couples; the laws of statics; and the geometry of continuous frames. The basic theory is simple; the difficulties arise in applying the principles to the many types of structures and loading conditions encountered. It is important to realize that unless the basic principles are thoroughly understood, their application to actual structures can be exceedingly difficult and often erroneous. Frequent review of good texts on analytical mechanics and structural theory will help fix these principles in the mind.

The equations of statics are so important that they are listed below for emphasis. For planar structures they are:

$$\Sigma H = 0 \qquad (6.3\text{-}1)$$

$$\Sigma V = 0 \qquad (6.3\text{-}2)$$

$$\Sigma M = 0 \qquad (6.3\text{-}3)$$

In words, these equations say that for a body to be in equilibrium, (1) the algebraic sum of the horizontal components of all forces acting on the body must equal zero; (2) the algebraic sum of the vertical components of all forces acting on the body must equal zero; and (3) the algebraic sum of the moments of all the forces acting on the body about any point in the plane of the forces must equal zero. These equations are necessary and sufficient for the solution of any statically determinate planar structure.

Free Body Diagrams. One of the most powerful tools of structural analysis is the free body diagram. A good designer will develop skill in their preparation and use; he will know and be able to apply the following steps:

1. Isolate a portion of the structure by passing a section (not necessarily straight) that cuts the member or members in which unknown forces or stresses are to be found.

2. Draw the free body diagram and show carefully all of the forces both internal and external acting on this free body.

3. Compute the unknown forces from the equations of statics.

Free body diagrams are also of value in the determination of the correct sense of shears, moments, rotations, and deflections; they give credence to the proverb that "a picture is worth 1000 words."

The use of free body diagrams will be illustrated numerous times in other sections of this handbook that deal with the design of specific structures.

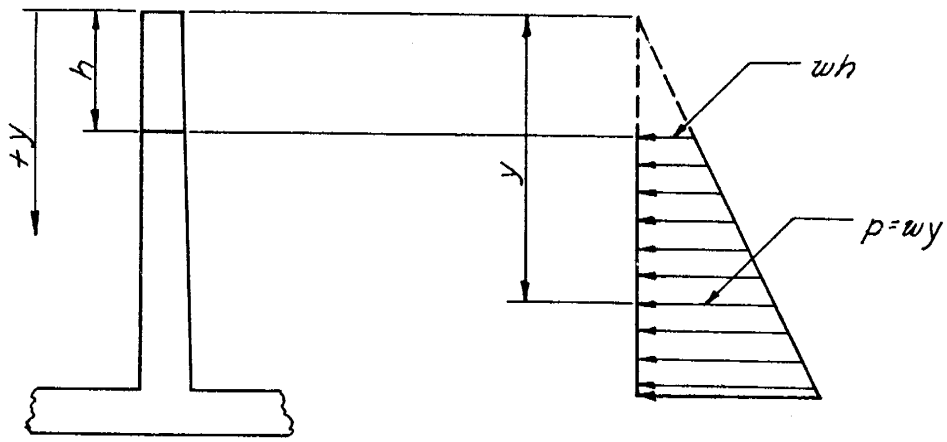
A designer's analytical ability can be lost through slovenly design notes and inadequate, sloppy engineering drawings. After a designer knows the fundamentals and how to apply them to specific problems, he must present his ideas in clear, neat, complete design notes and good drawings, or his effort and ability will not be recognized and appreciated. Anyone who has attempted to check a poorly prepared set of design notes or a carelessly prepared drawing will agree with the above comment. The reference value of cold, inadequately recorded engineering work is almost nil.

3.2 Shear and Moment Curves. Any of the modern textbooks on elementary structural theory and many books on strength of materials contain thorough discussions on the preparation of shear and moment curves and on the interrelationships between load, shear, and moment.

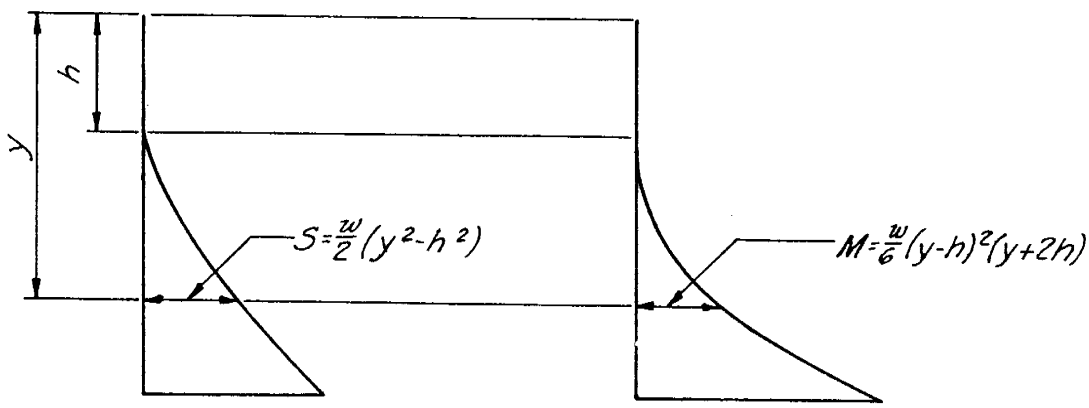
To aid in the computations involved in such work, several drawings have been prepared and included herein. Use of these drawings will facilitate the preparation of shear and moment curves on many of the cantilevers, beams, slabs, and rectangular frames encountered in soil conservation engineering. Your attention is especially directed to drawing ES-4, "Shear and Moments for Trapezoidal Load on Cantilever." The data contained in the three sheets of this drawing has found increasing use over the past 8 to 10 years since its original preparation.

Use of the following drawings will be illustrated in other sections of this handbook.

STRUCTURAL DESIGN: SHEAR AND MOMENTS FOR TRAPEZOIDAL LOAD ON CANTILEVER



CANTILEVER LOAD



SHEAR MOMENT

Consider one foot slice of cantilever.

h = depth of weir, height of surcharge, etc. in ft.
M = moment in ft.-lb. (for one foot slice).
p = pressure at depth y in lb. per sq. ft.
S = shear in lb. (for one foot slice)
w = weight of equivalent fluid in lb. per cu. ft.
y = vertical distance from top to any point in ft.

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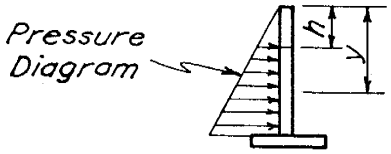
STRUCTURAL DESIGN: MOMENTS FOR TRAPEZOIDAL LOAD ON CANTILEVER



$$M = \frac{w}{6}(y-h)^2(y+2h)$$

$$w = 1$$

y \ h	0	1'-0"	1'-3"	1'-6"	1'-9"	2'-0"	2'-3"	2'-6"	2'-9"	3'-0"	3'-3"	3'-6"	3'-9"	4'-0"	4'-3"	4'-6"	4'-9"	5'-0"
1'-0"	0.17	0																
1'-6"	0.56	0.15	0.04	0														
2'-0"	1.33	0.67	0.42	0.21	0.06	0												
2'-6"	2.60	1.69	1.30	0.92	0.56	0.27	0.07	0										
3'-0"	4.50	3.33	2.81	2.25	1.69	1.17	0.70	0.33	0.09	0								
3'-3"	5.72	4.43	3.83	3.19	2.53	1.89	1.29	0.77	0.36	0.10	0							
3'-6"	7.15	5.73	5.06	4.33	3.57	2.81	2.08	1.42	0.84	0.40	0.10	0						
3'-9"	8.79	7.25	6.51	5.70	4.83	3.96	3.09	2.28	1.54	0.91	0.43	0.11	0					
4'-0"	10.7	9.0	8.19	7.29	6.33	5.33	4.34	3.38	2.47	1.67	0.98	0.46	0.12	0				
4'-3"	12.8	11.0	10.1	9.14	8.07	6.96	5.83	4.72	3.66	2.67	1.79	1.05	0.49	0.13	0			
4'-6"	15.2	13.3	12.3	11.3	10.1	8.85	7.59	6.33	5.10	3.94	2.86	1.92	1.13	0.52	0.14	0		
4'-9"	17.9	15.8	14.8	13.6	12.4	11.0	9.64	8.23	6.83	5.49	4.22	3.06	2.04	1.20	0.55	0.14	0	
5'-0"	20.8	18.7	17.6	16.3	15.0	13.5	12.0	10.4	8.86	7.33	5.87	4.50	3.26	2.17	1.27	0.58	0.15	0
5'-3"	24.1	21.8	20.7	19.3	17.9	16.3	14.6	12.9	11.2	9.49	7.83	6.25	4.78	3.45	2.29	1.34	0.61	0.16
5'-6"	27.7	25.3	24.1	22.7	21.1	19.4	17.6	15.8	13.9	12.0	10.1	8.33	6.64	5.06	3.65	2.42	1.41	0.65
5'-9"	31.7	29.1	27.8	26.3	24.7	22.9	20.9	18.9	16.9	14.8	12.8	10.8	8.83	7.02	5.34	3.84	2.54	1.48
6'-0"	36.0	33.3	32.0	30.4	28.6	26.7	24.6	22.5	20.2	18.0	15.8	13.5	11.4	9.33	7.40	5.63	4.04	2.67
6'-3"	40.7	37.9	36.5	34.8	32.9	30.9	28.7	26.4	24.0	21.6	19.1	16.7	14.3	12.0	9.83	7.78	5.91	4.23
6'-6"	45.8	42.9	41.3	39.6	37.6	35.4	33.1	30.7	28.1	25.5	22.9	20.3	17.6	15.1	12.7	10.3	8.17	6.19
6'-9"	51.3	48.2	46.6	44.8	42.7	40.4	38.0	35.4	32.7	29.9	27.1	24.2	21.4	18.6	15.9	13.3	10.8	8.55
7'.0"	57.2	54.0	52.3	50.4	48.2	45.8	43.2	40.5	37.6	34.7	31.6	28.6	25.5	22.5	19.5	16.7	13.9	11.3
7'-3"	63.5	60.2	58.5	56.5	54.2	51.7	49.0	46.1	43.0	39.9	36.7	33.4	30.1	26.8	23.6	20.5	17.4	14.6
7'-6"	70.3	66.9	65.1	63.0	60.6	57.9	55.1	52.1	48.9	45.6	42.1	38.7	35.2	31.6	28.2	24.8	21.4	18.2
7'-9"	77.6	74.0	72.2	70.0	67.5	64.7	61.8	58.6	55.2	51.7	48.1	44.4	40.7	36.9	33.2	29.5	25.9	22.4
8'-0"	85.3	81.7	79.7	77.5	74.9	72.0	68.9	65.5	62.0	58.3	54.5	50.6	46.7	42.7	38.7	34.7	30.8	27.0
8'-3"	93.6	89.8	87.8	85.4	82.7	79.8	76.5	73.0	69.3	65.5	61.5	57.3	53.2	48.9	44.7	40.4	36.2	32.1
8'-6"	102.4	98.4	96.4	93.9	91.1	88.0	84.6	81.0	77.2	73.1	68.9	64.6	60.2	55.7	51.2	46.7	42.2	37.8
8'-9"	111.7	107.6	105.5	102.9	100.0	96.8	93.3	89.5	85.5	81.3	76.9	72.4	67.7	63.0	58.2	53.4	48.7	43.9
9'-0"	121.5	117.3	115.1	112.5	109.5	106.2	102.5	98.6	94.4	90.0	85.4	80.7	75.8	70.8	65.8	60.8	55.7	50.7
9'-3"	131.9	127.6	125.3	122.6	119.5	116.1	112.3	108.2	103.9	99.3	94.5	89.5	84.4	79.2	74.0	68.6	63.3	58.0
9'-6"	142.9	138.5	136.1	133.3	130.1	126.6	122.6	118.4	113.9	109.1	104.2	99.0	93.7	88.2	82.7	77.1	71.4	65.8
9'-9"	154.5	149.9	147.5	144.6	141.3	137.6	133.6	129.2	124.5	119.6	114.4	109.0	103.5	97.8	92.0	86.1	80.2	74.3
10'-0"	166.7	162.0	159.5	156.5	153.1	149.3	145.2	140.6	135.8	130.7	125.3	119.7	113.9	108.0	101.9	95.8	89.6	83.3
10'-3"	179.5	174.7	172.1	169.1	165.6	161.6	157.3	152.7	147.7	142.4	136.8	131.0	125.0	118.8	112.5	106.1	99.6	93.0
10'-6"	192.9	188.0	185.4	182.4	178.6	174.6	170.2	165.3	160.2	154.7	148.9	142.9	136.7	130.3	123.7	117.0	110.2	103.4
10'-9"	207.0	202.0	199.3	196.1	192.4	188.2	183.6	178.7	173.3	167.7	161.7	155.5	149.0	142.4	135.6	128.6	121.5	114.3
11'-0"	221.8	216.7	213.9	210.6	206.8	202.5	197.8	192.7	187.2	181.3	175.2	168.8	162.1	155.2	148.1	140.8	133.5	126.0
11'-3"	237.3	232.0	229.2	225.8	221.9	217.5	212.6	207.4	201.7	195.7	189.3	182.7	175.8	168.6	161.3	153.8	146.1	138.3
11'-6"	253.5	248.1	245.1	241.7	237.7	233.1	228.2	222.8	216.9	210.7	204.2	197.3	190.2	182.8	175.2	167.4	159.5	151.4
11'-9"	270.4	264.8	261.8	258.3	254.2	249.5	244.4	238.9	232.9	226.5	219.8	212.7	205.3	197.7	189.8	181.8	173.5	165.2
12'-0"	288.0	282.3	279.3	275.6	271.4	266.7	261.4	255.7	249.6	243.0	236.1	228.8	221.2	213.3	205.2	196.9	188.3	179.7
12'-3"	306.4	300.6	297.5	293.7	289.4	284.5	279.2	273.3	267.0	260.3	253.1	245.6	237.8	229.7	221.3	212.7	203.9	194.9
12'-6"	325.5	319.6	316.4	312.6	308.2	303.2	297.7	291.7	285.2	278.3	270.9	263.3	255.2	246.9	238.2	229.3	220.2	210.9
12'-9"	345.4	339.4	336.1	332.2	327.7	322.6	317.0	310.8	304.2	297.1	289.6	281.6	273.4	264.8	255.9	246.7	237.3	227.7
13'-0"	366.2	360.0	356.7	352.7	348.0	342.8	337.1	330.8	323.9	316.7	309.0	300.8	292.3	283.5	274.3	264.9	255.2	245.3
13'-3"	387.7	381.4	378.0	373.9	369.2	363.9	358.0	351.5	344.5	337.1	329.2	320.8	312.1	303.0	293.6	283.9	273.9	263.7
13'-6"	410.1	403.6	400.1	396.0	391.2	385.7	379.7	373.1	365.9	358.3	350.2	341.7	332.7	323.4	313.7	303.8	293.5	283.0
13'-9"	433.3	426.7	423.2	418.9	414.0	408.4	402.3	395.5	388.2	380.4	372.1	363.3	354.1	344.6	334.7	324.4	313.9	303.1
14'-0"	457.3	450.7	447.0	442.7	437.7	432.0	425.7	418.8	411.3	403.3	394.8	385.9	376.5	366.7	356.5	346.0	335.1	324.0
14'-3"	482.3	475.5	471.8	467.4	462.2	456.4	450.0	443.0	435.3	427.1	418.5	409.3	399.7	389.6	379.2	368.4	357.2	345.8
14'-6"	508.1	501.2	497.4	492.9	487.7	481.8	475.2	468.0	460.2	451.9	443.0	433.6	423.7	413.4	402.7	391.7	380.3	368.5
14'-9"	534.8	527.8	524.0	519.4	514.0	508.0	501.6	494.0	486.0	477.5	468.4	458.8	448.7	438.2	427.2	415.9	404.2	392.1
15'-0"	562.5	555.3	551.4	546.8	541.3	535.2	528.3	520.8	512.7	504.0	494.7	484.9	474.6	463.8	452.6	441.0	429.0	416.7

REFERENCE

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

ENGINEERING STANDARDS UNIT

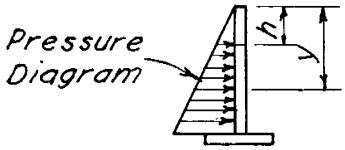
STANDARD DWG. NO.
ES - 4

SHEET 2 OF 3

DATE 11 - 1 - 49

Revised 6-4-51

STRUCTURAL DESIGN: SHEAR FOR TRAPEZOIDAL LOAD ON CANTILEVER



$$S = \frac{w}{2}(y^2 - h^2)$$

$$w = 1$$

y \ h	0	1'-0"	1'-3"	1'-6"	1'-9"	2'-0"	2'-3"	2'-6"	2'-9"	3'-0"	3'-3"	3'-6"	3'-9"	4'-0"	4'-3"	4'-6"	4'-9"	5'-0"
1'-0"	0.50	0																
1'-6"	1.13	0.63	0.34	0														
2'-0"	2.00	1.50	1.22	0.88	0.47	0												
2'-6"	3.13	2.63	2.34	2.00	1.59	1.13	0.59	0										
3'-0"	4.50	4.00	3.72	3.38	2.97	2.50	1.97	1.38	0.72	0								
3'-3"	5.28	4.78	4.50	4.15	3.75	3.28	2.75	2.16	1.50	0.78	0							
3'-6"	6.13	5.63	5.34	5.00	4.59	4.13	3.59	3.00	2.34	1.63	0.84	0						
3'-9"	7.03	6.53	6.25	5.90	5.50	5.03	4.50	3.90	3.25	2.53	1.75	0.90	0					
4'-0"	8.00	7.50	7.22	6.88	6.47	6.00	5.47	4.88	4.22	3.50	2.72	1.88	0.97	0				
4'-3"	9.03	8.53	8.25	7.90	7.50	7.03	6.50	5.90	5.25	4.53	3.75	2.90	2.00	1.03	0			
4'-6"	10.1	9.63	9.34	9.00	8.59	8.13	7.59	7.00	6.34	5.63	4.84	4.00	3.09	2.13	1.09	0		
4'-9"	11.3	10.8	10.5	10.2	9.75	9.28	8.75	8.16	7.50	6.78	6.00	5.16	4.25	3.28	2.25	1.16	0	
5'-0"	12.5	12.0	11.7	11.4	11.0	10.5	9.97	9.38	8.72	8.00	7.22	6.38	5.47	4.50	3.47	2.38	1.22	0
5'-3"	13.8	13.3	13.0	12.7	12.3	11.8	11.3	10.7	10.0	9.28	8.50	7.66	6.75	5.78	4.75	3.66	2.50	1.28
5'-6"	15.1	14.6	14.3	14.0	13.6	13.1	12.6	12.0	11.3	10.6	9.84	9.00	8.09	7.13	6.09	5.00	3.84	2.63
5'-9"	16.5	16.0	15.8	15.4	15.0	14.5	14.0	13.4	12.8	12.0	11.3	10.4	9.50	8.53	7.50	6.40	5.25	4.03
6'-0"	18.0	17.5	17.2	16.9	16.5	16.0	15.5	14.9	14.2	13.5	12.7	11.9	11.0	10.0	8.97	7.88	6.72	5.50
6'-3"	19.5	19.0	18.8	18.4	18.0	17.5	17.0	16.4	15.8	15.0	14.3	13.4	12.5	11.5	10.5	9.41	8.25	7.00
6'-6"	21.1	20.6	20.3	20.0	19.6	19.1	18.6	18.0	17.3	16.6	15.8	15.0	14.1	13.1	12.1	11.0	9.84	8.63
6'-9"	22.8	22.3	22.0	21.7	21.3	20.8	20.3	19.7	19.0	18.3	17.5	16.7	15.8	14.8	13.8	12.7	11.5	10.3
7'-0"	24.5	24.0	23.7	23.4	23.0	22.5	22.0	21.4	20.7	20.0	19.2	18.4	17.5	16.5	15.5	14.4	13.2	12.0
7'-3"	26.3	25.8	25.5	25.2	24.8	24.3	23.8	23.2	22.5	21.8	21.0	20.2	19.3	18.3	17.3	16.2	15.0	13.8
7'-6"	28.1	27.6	27.3	27.0	26.6	26.1	25.6	25.0	24.3	23.6	22.8	22.0	21.1	20.1	19.1	18.0	16.8	15.6
7'-9"	30.0	29.5	29.3	28.9	28.5	28.0	27.5	26.9	26.3	25.5	24.8	23.9	23.0	22.0	21.0	19.9	18.8	17.5
8'-0"	32.0	31.5	31.2	30.9	30.5	30.0	29.5	28.9	28.2	27.5	26.7	25.9	25.0	24.0	23.0	21.9	20.7	19.5
8'-3"	34.0	33.5	33.3	32.9	32.5	32.0	31.5	30.9	30.3	29.5	28.8	27.9	27.0	26.0	25.0	23.9	22.8	21.5
8'-6"	36.1	35.6	35.3	35.0	34.6	34.1	33.6	33.0	32.3	31.6	30.8	30.0	29.1	28.1	27.1	26.0	24.8	23.6
8'-9"	38.3	37.8	37.5	37.2	36.8	36.3	35.8	35.2	34.5	33.8	33.0	32.2	31.3	30.3	29.3	28.2	27.0	25.8
9'-0"	40.5	40.0	39.7	39.4	39.0	38.5	38.0	37.4	36.7	36.0	35.2	34.4	33.5	32.5	31.5	30.4	29.2	28.0
9'-3"	42.8	42.3	42.0	41.7	41.3	40.8	40.3	39.7	39.0	38.3	37.5	36.7	35.8	34.8	33.8	32.7	31.5	30.3
9'-6"	45.1	44.6	44.3	44.0	43.6	43.1	42.6	42.0	41.3	40.6	39.8	39.0	38.1	37.1	36.1	35.0	33.8	32.6
9'-9"	47.5	47.0	46.8	46.4	46.0	45.5	45.0	44.4	43.8	43.0	42.3	41.4	40.5	39.5	38.5	37.4	36.3	35.0
10'-0"	50.0	49.5	49.2	48.9	48.5	48.0	47.5	46.9	46.2	45.5	44.7	43.9	43.0	42.0	41.0	39.9	38.7	37.5
10'-3"	52.5	52.0	51.8	51.4	51.0	50.5	50.0	49.4	48.8	48.0	47.3	46.4	45.5	44.5	43.5	42.4	41.3	40.0
10'-6"	55.1	54.6	54.3	54.0	53.6	53.1	52.6	52.0	51.3	50.6	49.8	49.0	48.1	47.1	46.1	45.0	43.8	42.6
10'-9"	57.8	57.3	57.0	56.7	56.3	55.8	55.3	54.7	54.0	53.3	52.5	51.7	50.8	49.8	48.8	47.7	46.5	45.3
11'-0"	60.5	60.0	59.7	59.4	59.0	58.5	58.0	57.4	56.7	56.0	55.2	54.4	53.5	52.5	51.5	50.4	49.2	48.0
11'-3"	63.3	62.8	62.5	62.2	61.8	61.3	60.8	60.2	59.5	58.8	58.0	57.2	56.3	55.3	54.3	53.2	52.0	50.8
11'-6"	66.1	65.6	65.3	65.0	64.6	64.1	63.6	63.0	62.3	61.6	60.8	60.0	59.1	58.1	57.1	56.0	54.8	53.6
11'-9"	69.0	68.5	68.3	67.9	67.5	67.0	66.5	65.9	65.3	64.5	63.8	62.9	62.0	61.0	60.0	58.9	57.8	56.5
12'-0"	72.0	71.5	71.2	70.9	70.5	70.0	69.5	68.9	68.2	67.5	66.7	65.9	65.0	64.0	63.0	61.9	60.7	59.5
12'-3"	75.0	74.5	74.3	73.9	73.5	73.0	72.5	71.9	71.3	70.5	69.8	68.9	68.0	67.0	66.0	64.9	63.8	62.5
12'-6"	78.1	77.6	77.3	77.0	76.6	76.1	75.6	75.0	74.3	73.6	72.8	72.0	71.1	70.1	69.1	68.0	66.8	65.6
12'-9"	81.3	80.8	80.5	80.2	79.8	79.3	78.8	78.2	77.5	76.8	76.0	75.2	74.3	73.3	72.3	71.2	70.0	68.8
13'-0"	84.5	84.0	83.7	83.4	83.0	82.5	82.0	81.4	80.7	80.0	79.2	78.4	77.5	76.5	75.5	74.4	73.2	72.0
13'-3"	87.8	87.3	87.0	86.7	86.3	85.8	85.3	84.7	84.0	83.3	82.5	81.7	80.8	79.8	78.8	77.7	76.5	75.3
13'-6"	91.1	90.6	90.3	90.0	89.6	89.1	88.6	88.0	87.3	86.6	85.8	85.0	84.1	83.1	82.1	81.0	79.8	78.6
13'-9"	94.5	94.0	93.8	93.4	93.0	92.5	92.0	91.4	90.8	90.0	89.3	88.4	87.5	86.5	85.5	84.4	83.3	82.0
14'-0"	98.0	97.5	97.2	96.9	96.5	96.0	95.5	94.9	94.2	93.5	92.7	91.9	91.0	90.0	89.0	87.9	86.7	85.5
14'-3"	101.5	101.0	100.8	100.4	100.0	99.5	99.0	98.4	97.8	97.0	96.3	95.4	94.5	93.5	92.5	91.4	90.3	89.0
14'-6"	105.1	104.6	104.3	104.0	103.6	103.1	102.6	102.0	101.3	100.6	99.8	99.0	98.1	97.1	96.1	95.0	93.8	92.6
14'-9"	108.8	108.3	108.0	107.7	107.3	106.8	106.3	105.7	105.0	104.3	103.5	102.7	101.8	100.8	99.8	98.7	97.5	96.3
15'-0"	112.5	112.0	111.7	111.4	111.0	110.5	110.0	109.4	108.7	108.0	107.2	106.4	105.5	104.5	103.5	102.4	101.2	100.0

REFERENCE

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

ENGINEERING STANDARDS UNIT

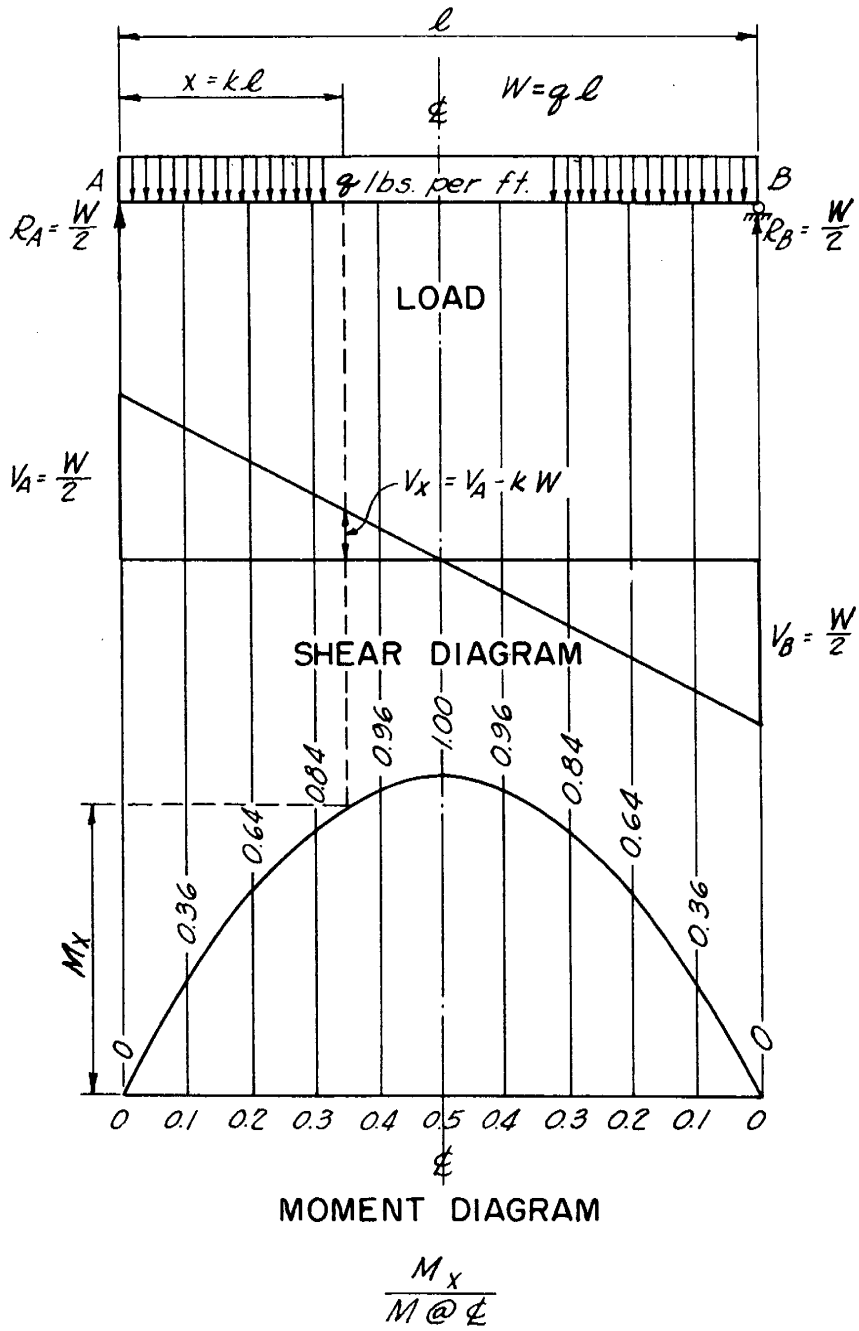
STANDARD DWG. NO.
ES - 4
SHEET 3 OF 3
DATE 11-1-49

USDA-SCS-HYATTSVILLE MD 1967 Revised 6-4-51

STRUCTURAL DESIGN: SIMPLE BEAM MOMENTS FOR UNIFORMLY DISTRIBUTED LOAD

$$M_x = M_{k\ell} = \frac{q\ell^2}{2} k(1-k) = \frac{W\ell}{2} k(1-k)$$

$$\text{or } M_x = CW\ell, \text{ where } C = \frac{k}{2}(1-k)$$



k	C	$\frac{M_x}{M@℄}$
0.01	0.00495	0.0396
0.02	0.00980	0.0784
0.03	0.01455	0.1164
0.04	0.01920	0.1536
0.05	0.02375	0.1900
0.06	0.02820	0.2256
0.07	0.03255	0.2604
0.08	0.03680	0.2944
0.09	0.04095	0.3276
0.1	0.04500	0.3600
0.11	0.04895	0.3916
0.12	0.05280	0.4224
0.13	0.05655	0.4524
0.14	0.06020	0.4816
0.15	0.06375	0.5100
0.16	0.06720	0.5376
0.17	0.07055	0.5644
0.18	0.07380	0.5904
0.19	0.07695	0.6156
0.2	0.08000	0.6400
0.21	0.08295	0.6636
0.22	0.08580	0.6864
0.23	0.08855	0.7084
0.24	0.09120	0.7296
0.25	0.09375	0.7500
0.26	0.09620	0.7696
0.27	0.09855	0.7884
0.28	0.10080	0.8064
0.29	0.10295	0.8236
0.3	0.10500	0.8400
0.31	0.10695	0.8556
0.32	0.10880	0.8704
0.33	0.11055	0.8844
0.34	0.11220	0.8976
0.35	0.11375	0.9100
0.36	0.11520	0.9216
0.37	0.11655	0.9324
0.38	0.11780	0.9424
0.39	0.11895	0.9516
0.4	0.12000	0.9600
0.41	0.12095	0.9676
0.42	0.12180	0.9744
0.43	0.12255	0.9804
0.44	0.12320	0.9856
0.45	0.12375	0.9900
0.46	0.12420	0.9936
0.47	0.12455	0.9964
0.48	0.12480	0.9984
0.49	0.12495	0.9996
0.5	0.12500	1.00

REFERENCE

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

ENGINEERING STANDARDS UNIT

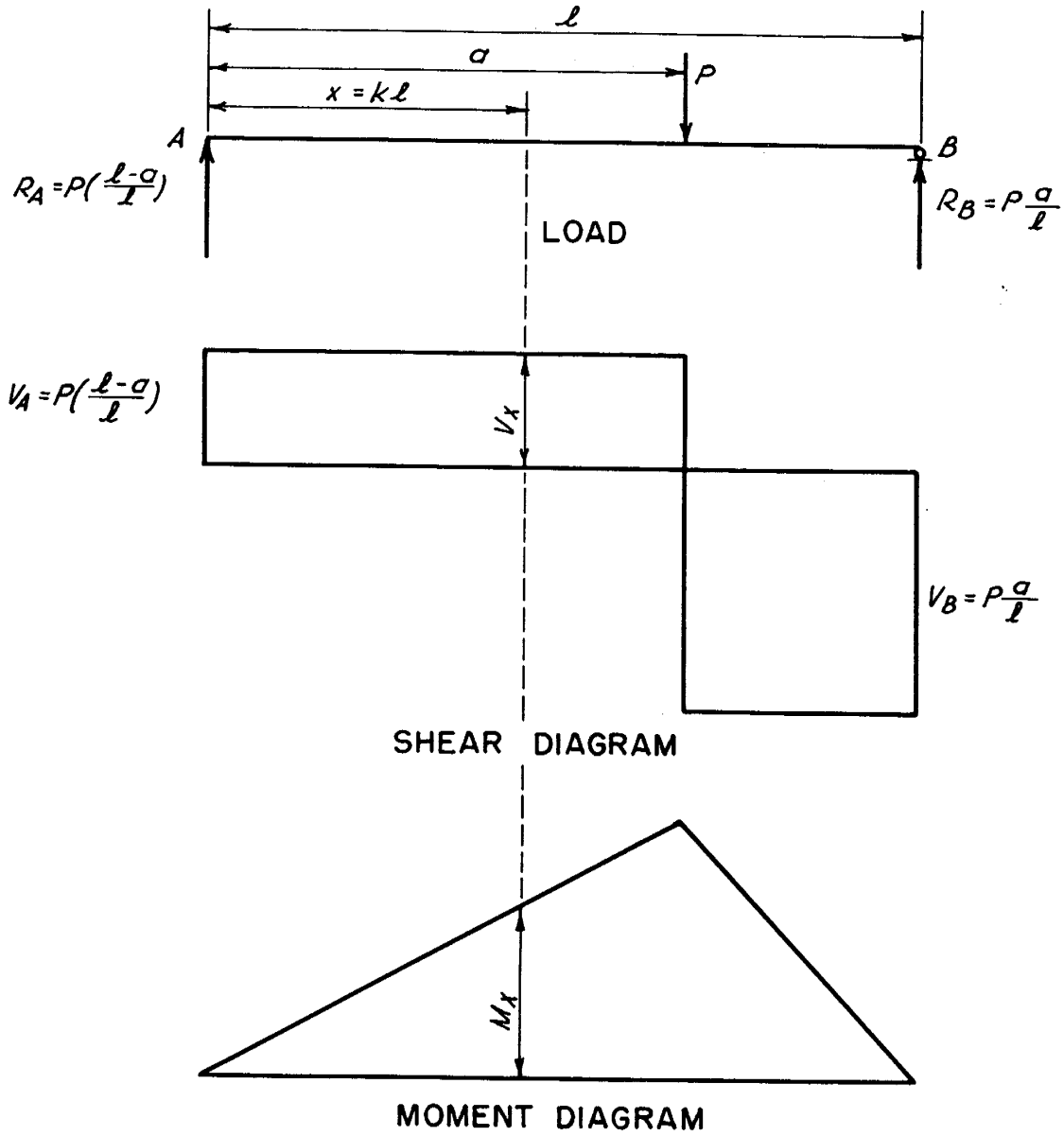
STANDARD DWG. NO. ES - 1

SHEET 1 OF 1

DATE 10 - 26 - 49

REVISED 3-30-50

STRUCTURAL DESIGN: SIMPLE BEAM MOMENTS FOR CONCENTRATED LOAD



$M_x = P(\ell - a)k$, when $k\ell < a$; $M_x = Pa(1-k)$ when $k\ell > a$

$$M_{max} = P(\frac{\ell - a}{\ell})a$$

When load (P) is at ℄ of beam ($a = \frac{\ell}{2}$)

$R_A = R_B = \frac{P}{2}$; $V_{max} = \frac{P}{2}$

$M_x = \frac{P}{2}k\ell$; $M_{max} = \frac{P\ell}{4}$

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING STANDARDS UNIT	STANDARD DWG. NO. ES-2 SHEET 1 OF 1 DATE 11-1-49

STRUCTURAL DESIGN: SIMPLE BEAM MOMENTS FOR TRAPEZOIDAL LOAD

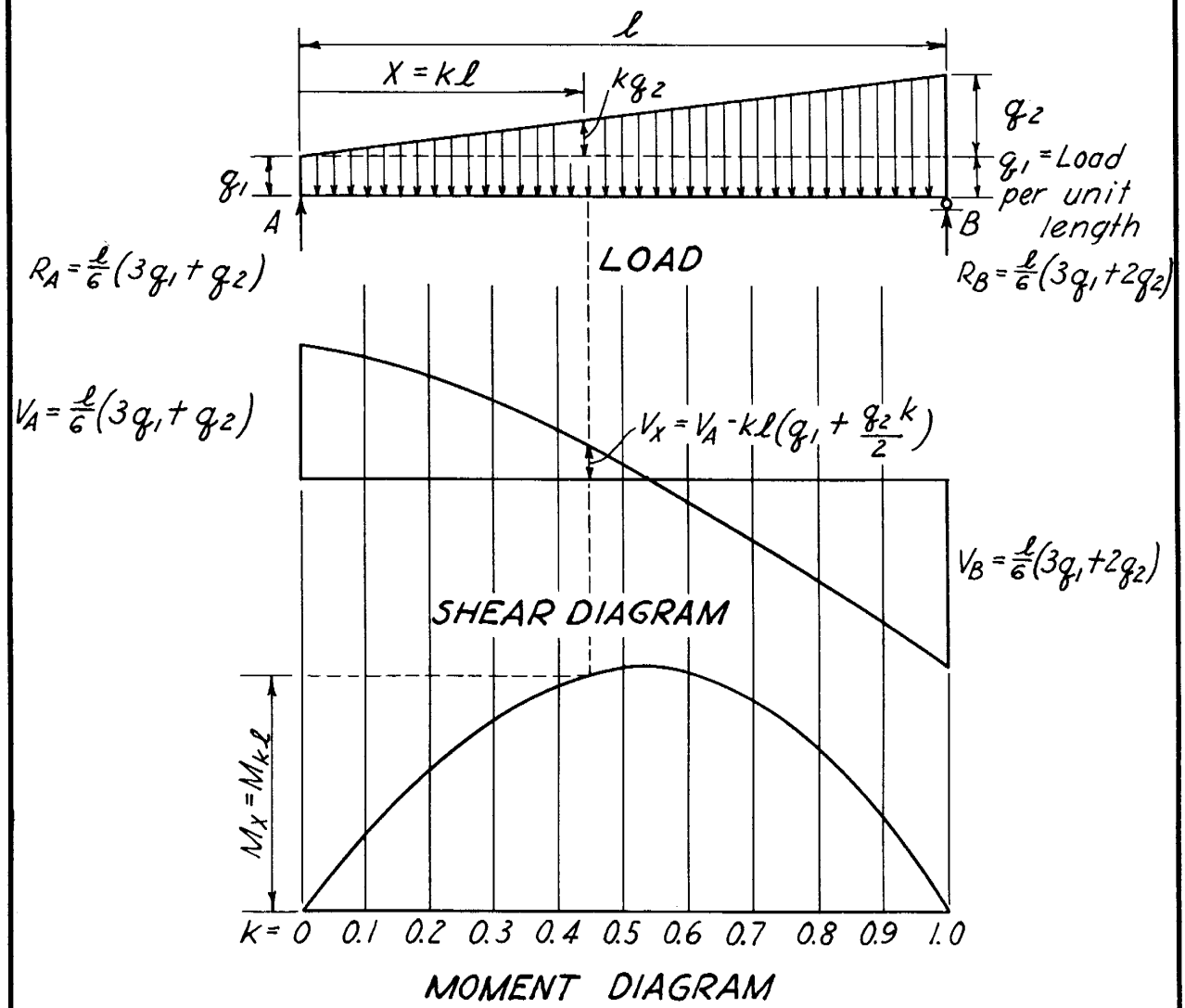
$$M_x = M_{k\ell} = \frac{q_1 \ell^2}{6} \cdot D(E + C)$$

where $C = \frac{q_2}{q_1}$, $D = k(1-k^2)$, $E = \frac{3}{1+k}$

M max. when $k = \frac{1}{c}\left[-1 + \sqrt{\frac{c^2+3c+3}{3}}\right]$

NOTE: Ordinarily it is not necessary to compute the maximum moment. It can usually be determined from the moment diagram with sufficient accuracy. The maximum moment will occur between $k = 0.500$ and $k = 0.577$

k	D	E
0.1	0.099	2.727
0.2	0.192	2.500
0.3	0.273	2.308
0.4	0.336	2.143
0.5	0.375	2.000
0.6	0.384	1.875
0.7	0.357	1.765
0.8	0.288	1.666
0.9	0.171	1.579



REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING STANDARDS UNIT	STANDARD DWG. NO. ES-3 SHEET 1 OF 1 DATE 11-1-49

FED 3-30-5

STRUCTURAL DESIGN: SIMPLE BEAM MOMENTS FOR TRIANGULAR LOAD

$$M_x = M_{k\ell} = \tfrac{1}{6} q \ell^2 (k - k^3) = \tfrac{1}{3} W \ell (k - k^3)$$

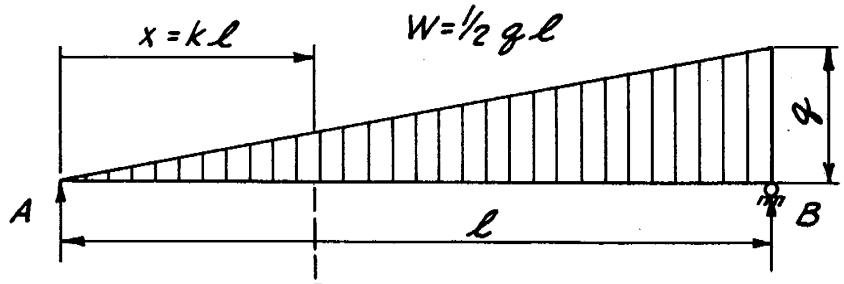
$$C = \tfrac{1}{3}(k - k^3)$$

$$M_{k\ell} = CW\ell$$

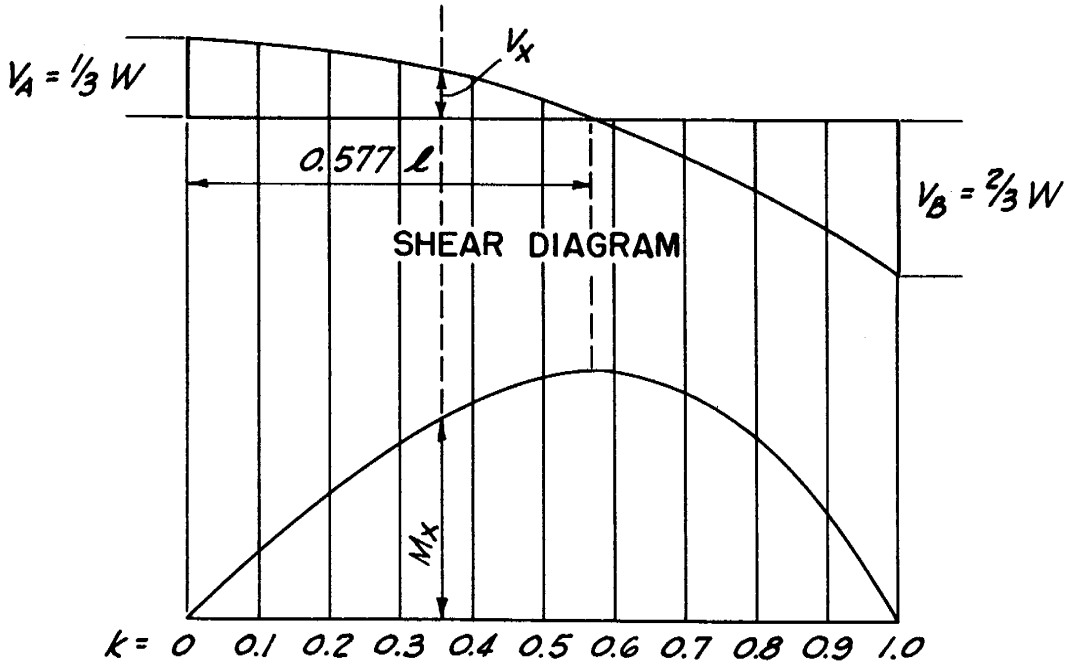
M is maximum when $k = 0.577$

$$V_x = V_{k\ell} = W(\tfrac{1}{3} - k^2)$$

k	C
0.1	0.033
0.2	0.064
0.3	0.091
0.4	0.112
0.5	0.125
0.577	0.1283
0.6	0.128
0.7	0.119
0.8	0.096
0.9	0.057



LOAD DIAGRAM



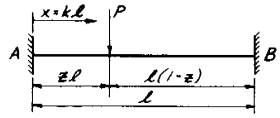
SHEAR DIAGRAM

MOMENT DIAGRAM

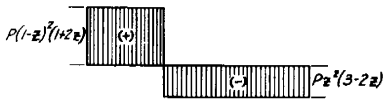
REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE	STANDARD DWG. NO.
Continuity in Concrete Building Frames, Third Edition, Portland Cement Association.	ENGINEERING STANDARDS UNIT	ES-23
		SHEET 1 OF 1
		DATE 4-1-50

STRUCTURAL DESIGN: FIXED ENDED BEAM MOMENTS FOR CONCENTRATED LOAD, UNIFORMLY DISTRIBUTED LOAD, AND HYDROSTATIC LOAD ON PRISMATIC BEAMS

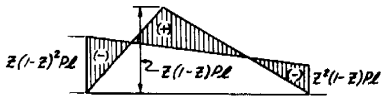
CONCENTRATED LOAD



Load Diagram



Shear Diagram



Moment Diagram

MOMENT AT ANY POINT kℓ FROM A

For $0 \le k \le z$

$M_x = M_{k\ell} = (1-z)^2(k+2zk-z)P\ell$

For $z \le k \le 1$

$M_x = M_{k\ell} = z^2(2zk-z+2-3k)P\ell$

MAXIMUM POSITIVE MOMENT (k = z)

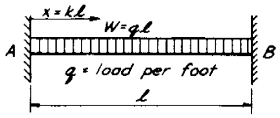
$M_{z\ell} = 2z^2(1-z)^2 P\ell$

FIXED END MOMENTS (Negative)

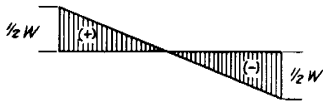
$M^F_{AB} = z(1-z)^2 P\ell$

$M^F_{BA} = z^2(1-z)P\ell$

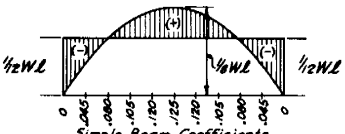
UNIFORMLY DISTRIBUTED LOAD



Load Diagram



Shear Diagram



Simple Beam Coefficients at tenth points in M = CWℓ

Moment Diagram

SHEAR AT ANY POINT kℓ FROM A

$V_x = V_{k\ell} = \frac{1}{2}(1-2k)W$

MOMENT AT ANY POINT kℓ FROM A

$M_x = M_{k\ell} = \frac{1}{12}(6k-6k^2-1)W\ell$

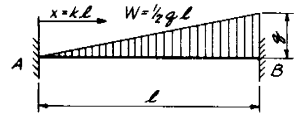
MAXIMUM POSITIVE MOMENT (k = ½)

$M_{\frac{1}{2}\ell} = \frac{1}{24}W\ell$

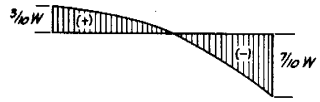
FIXED END MOMENTS (Negative)

$M^F_{AB} = M^F_{BA} = \frac{1}{12}W\ell$

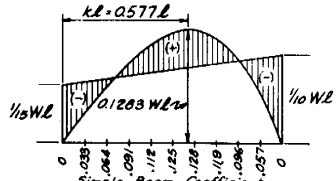
HYDROSTATIC LOAD



Load Diagram



Shear Diagram



Simple Beam Coefficients at tenth points in M = CWℓ

Moment Diagram

SHEAR AT ANY POINT kℓ FROM A

$V_x = V_{k\ell} = \frac{1}{10}(3-10k^2)W$

MOMENT AT ANY POINT kℓ FROM

$M_x = M_{k\ell} = \frac{1}{30}(9k-2-10k^3)W\ell$

MAXIMUM POSITIVE MOMENT (k = 0.5477)

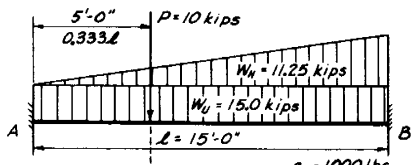
$M_{0.5477\ell} = 0.0429\,W\ell$

FIXED END MOMENTS (Negative)

$M^F_{AB} = \frac{1}{15}W\ell$

$M^F_{BA} = \frac{1}{10}W\ell$

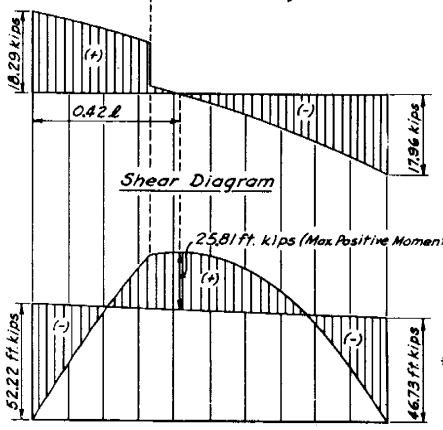
EXAMPLE



Load Diagram

$q_U = 1{,}000$ lbs.

$q_H = 1{,}500$ lbs.



Shear Diagram

Moment Diagram

SHEAR IN KIPS

	0	0.1ℓ	0.2ℓ	0.3ℓ	0.333ℓ	0.4ℓ	0.5ℓ	0.6ℓ	0.7ℓ	0.8ℓ	0.9ℓ	ℓ
Conc. Load	+7.41	+7.41	+7.41	+7.41	+7.41 / -2.59	-2.59	-2.59	-2.59	-2.59	-2.59	-2.59	-2.59
** U.L = ½(1-2k)W_U	+7.50	+6.00	+4.50	+3.00	+2.50	+1.50	0	-1.50	-3.00	-4.50	-6.00	-7.50
** H.L = 1/10(3-10k²)W_H	+3.38	+3.26	+2.92	+2.36	+2.13	+1.58	+0.56	-0.68	-2.14	-3.83	-5.74	-7.87
Total	+18.29	+16.67	+14.83	+12.77	+12.04 / +2.04	+0.49	-2.03	-4.77	-7.73	-10.92	-14.33	-17.96

FIXED END MOMENTS

Conc. Load - $M_A = z(1-z)^2 P\ell = 0.333(0.667)^2 10 \times 15 =$ 22.22 ft. kips

Unif. Load - $M_A = \frac{1}{12} W_U \ell = \frac{1}{12} \times 15 \times 15 =$ 18.75 ft. kips

Hyd. Load - $M_A = \frac{1}{15} W_H \ell = \frac{1}{15} \times 11.25 \times 15 =$ 11.25 ft. kips

Total $M^F_{AB} =$ 52.22 ft. kips

Conc. Load - $M_B = z^2(1-z)P\ell = (0.333)^2(0.667)10 \times 15 =$ 11.11 ft. kips

Unif. Load - $M_B = \frac{1}{12} W_U \ell = \frac{1}{12} \times 15 \times 15 =$ 18.75 ft. kips

Hyd. Load - $M_B = \frac{1}{10} W_H \ell = \frac{1}{10} \times 11.25 \times 15 =$ 16.87 ft. kips

Total $M^F_{BA} =$ 46.73 ft. kips

SIMPLE BEAM MOMENTS IN FT. KIPS

	0	0.1ℓ	0.2ℓ	0.3ℓ	0.333ℓ	0.4ℓ	0.5ℓ	0.6ℓ	0.7ℓ	0.8ℓ	0.9ℓ	ℓ
Conc. Load	0	10.00	20.00	30.00	33.33	30.00	25.00	20.00	15.00	10.00	5.00	0
** U.L = C_U W_U ℓ	0	10.13	18.00	23.63	25.00	27.00	28.13	27.00	23.63	18.00	10.13	0
** H.L = C_H W_H ℓ	0	5.57	10.80	15.36	16.67	18.90	21.10	21.60	20.10	16.20	9.62	0
Total	0	25.70	48.80	68.99	75.00	75.90	74.23	68.60	58.73	44.20	24.75	0

* See ES-1, ES-2, and ES-23

** The Uniform load and Hydrostatic load can be combined and ES-3 used.

REFERENCE Continuity in Concrete Building Frames, Third Edition, Portland Cement Association.

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

ENGINEERING STANDARDS UNIT

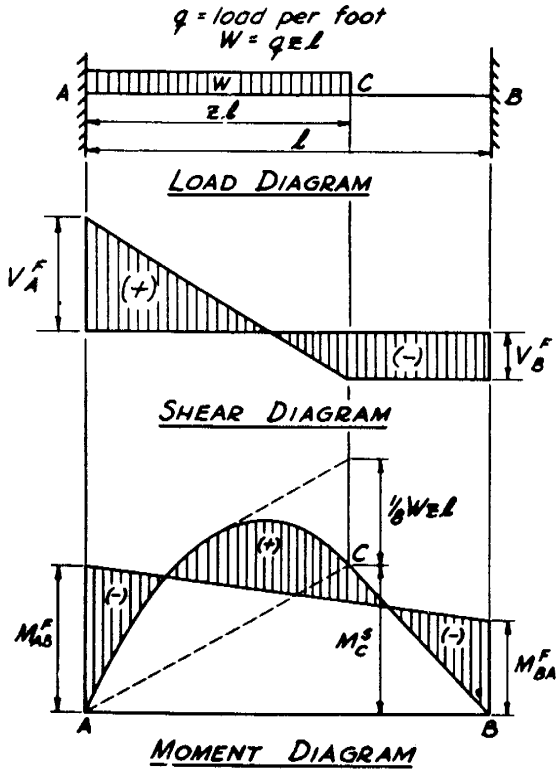
STANDARD DWG. NO. ES-17

SHEET 1 OF 1

DATE 4-11-50

Revised 6-4-51

STRUCTURAL DESIGN: FIXED ENDED BEAM MOMENTS FOR PARTIAL UNIFORMLY DISTRIBUTED LOAD— PRISMATIC BEAMS



FIXED END MOMENTS

$M_{AB}^F = \frac{1}{12} l z (6 - 8z + 3z^2) W$

$M_{BA}^F = \frac{1}{12} l z^2 (4 - 3z) W$

SIMPLE BEAM REACTIONS

$R_A^S = V_A^S = \frac{1}{2} W (2 - z)$

$R_B^S = V_B^S = \frac{1}{2} W z$

FIXED END REACTIONS

$$R_A^F = V_A^F = R_A^S + \frac{(M_{AB}^F - M_{BA}^F)}{l}$$

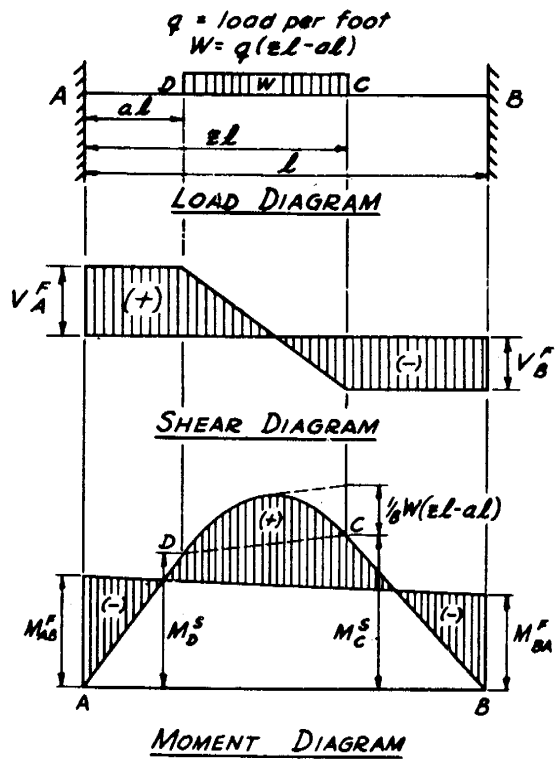
$$R_B^F = V_B^F = R_B^S - \frac{(M_{AB}^F - M_{BA}^F)}{l}$$

SIMPLE BEAM MOMENT AT C

$M_C^S = R_B^S (l - zl)$

PLOTTING MOMENT DIAGRAM

1. Plot M_C^S at C
2. Plot simple beam moment diagram for span (zl) and load (W) vertically above line AC (See ES-1)
3. Plot M_{AB}^F at A
4. Plot M_{BA}^F at B



FIXED END MOMENTS

$M_{AB}^F = \frac{1}{12} l [z(6 - 8z + 3z^2) q z l - a^2 (6 - 8a + 3a^2) q l]$

$M_{BA}^F = \frac{1}{12} l [z^2 (4 - 3z) q z l - a^3 (4 - 3a) q l]$

SIMPLE BEAM REACTIONS

$R_B^S = V_B^S = \frac{1}{2} W (a + z)$

$R_A^S = V_A^S = \frac{1}{2} W (2 - a - z)$

FIXED END REACTIONS

$$R_A^F = V_A^F = R_A^S + \frac{(M_{AB}^F - M_{BA}^F)}{l}$$

$$R_B^F = V_B^F = R_B^S - \frac{(M_{AB}^F - M_{BA}^F)}{l}$$

SIMPLE BEAM MOMENTS AT C & D

$M_C^S = R_B^S (l - zl)$

$M_D^S = R_A^S (al)$

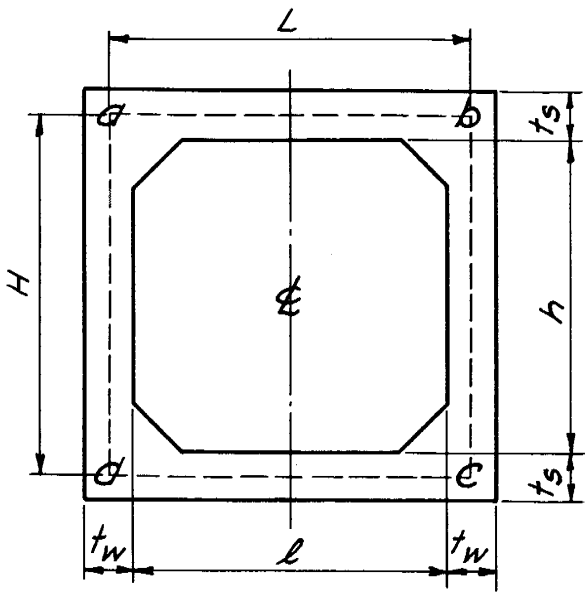
PLOTTING MOMENT DIAGRAM

1. Plot M_C^S at C
2. Plot M_D^S at D
3. Plot simple beam moment diagram for span ($zl - al$) and load (W) vertically above line CD (See ES-1)
4. Plot M_{AB}^F at A
5. Plot M_{BA}^F at B

REFERENCE Continuity in Concrete Building Frames, Third Edition, Portland Cement Association.	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING STANDARDS UNIT	STANDARD DWG. NO. ES-32 SHEET 1 OF 1 DATE 5-26-50

Revised 6-4-51

STRUCTURAL DESIGN: MOMENTS IN SINGLE BARREL



Assumptions -

1 - Loads and frame are symmetrical about center line of frame.

2 - The analysis is based on center line dimensions.

3 - The effect of small fillets is neglected.

$$H = h + t_s$$

$$L = \ell + t_w$$

$$\frac{K_s}{K_w} = C = \left(\frac{t_s}{t_w}\right)^3 \cdot \frac{H}{L}$$

$$p = \frac{C}{(C+2)^2 - 1}$$

$$q = C + 2$$

$$U_a^F = M_{ab}^F + M_{ad}^F$$

$$U_c^F = M_{cd}^F + M_{cb}^F$$

$$M_{ab} = M_{ab}^F - p(U_c^F + q\,U_a^F)$$

$$M_{dc} = M_{dc}^F + p(U_a^F + q\,U_c^F)$$

$$M_{dc} + M_{da} = 0$$

$$M_{ab} + M_{ad} = 0$$

Sign convention - A moment acting in a clockwise direction on a joint is positive.

Caution: Use proper sign of fixed end moments when substituting them in the given equations.

Nomenclature:

M_{ab} = Moment at end "a" of member "ab".

M_{ab}^F = Fixed end moment at end "a" of member "ab".

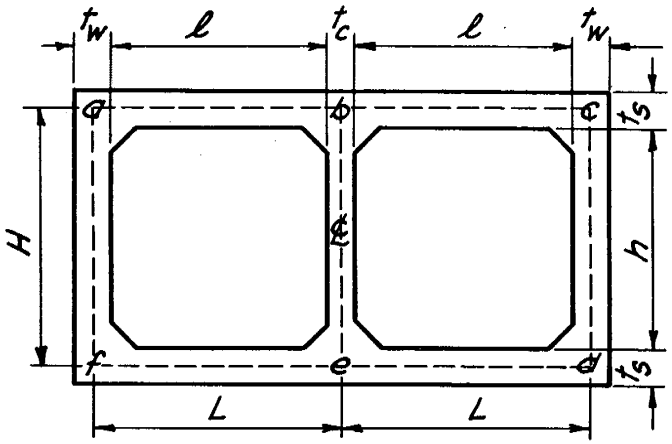
U_a^F = Algebraic sum of fixed end moments at a = unbalanced fixed end moment at a.

K_w = Stiffness of wall.

K_s = Stiffness of slab.

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING STANDARDS UNIT	STANDARD DWG. NO. ES-28 SHEET 1 OF 1 DATE 5-23-50

STRUCTURAL DESIGN: MOMENTS IN DOUBLE BARREL



Assumptions:

1. - Loads and frame are symmetrical about center line of frame.
2. - The analysis is based on center line dimensions.
3. - The effect of small fillets is neglected.

$$M_{ab} = M_{ab}^F + 2m\left(U_f^F - nU_a^F\right)$$
$$M_{ba} = M_{ba}^F + m\left(U_f^F - nU_a^F\right)$$
$$M_{ef} = M_{ef}^F + m\left(U_a^F - nU_f^F\right)$$
$$M_{fe} = M_{fe}^F + 2m\left(U_a^F - nU_f^F\right)$$
$$M_{ab} + M_{af} = 0;\ M_{fa} + M_{fe} = 0;$$
$$M_{be} = M_{eb} = 0.$$

$$H = h + t_s$$

$$L = \ell + \frac{t_w}{2} + \frac{t_c}{2}$$

$$\frac{K_s}{K_w} = C = \left(\frac{t_s}{t_w}\right)^3 \cdot \frac{H}{L}$$

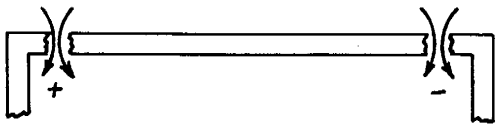
$$m = \frac{C}{4(C+1)^2 - 1}$$

$$n = 2(C+1)$$

$$U_f^F = M_{fa}^F + M_{fe}^F$$

$$U_a^F = M_{ab}^F + M_{af}^F$$

Sign Convention - A moment acting in a clockwise direction on a joint is positive.



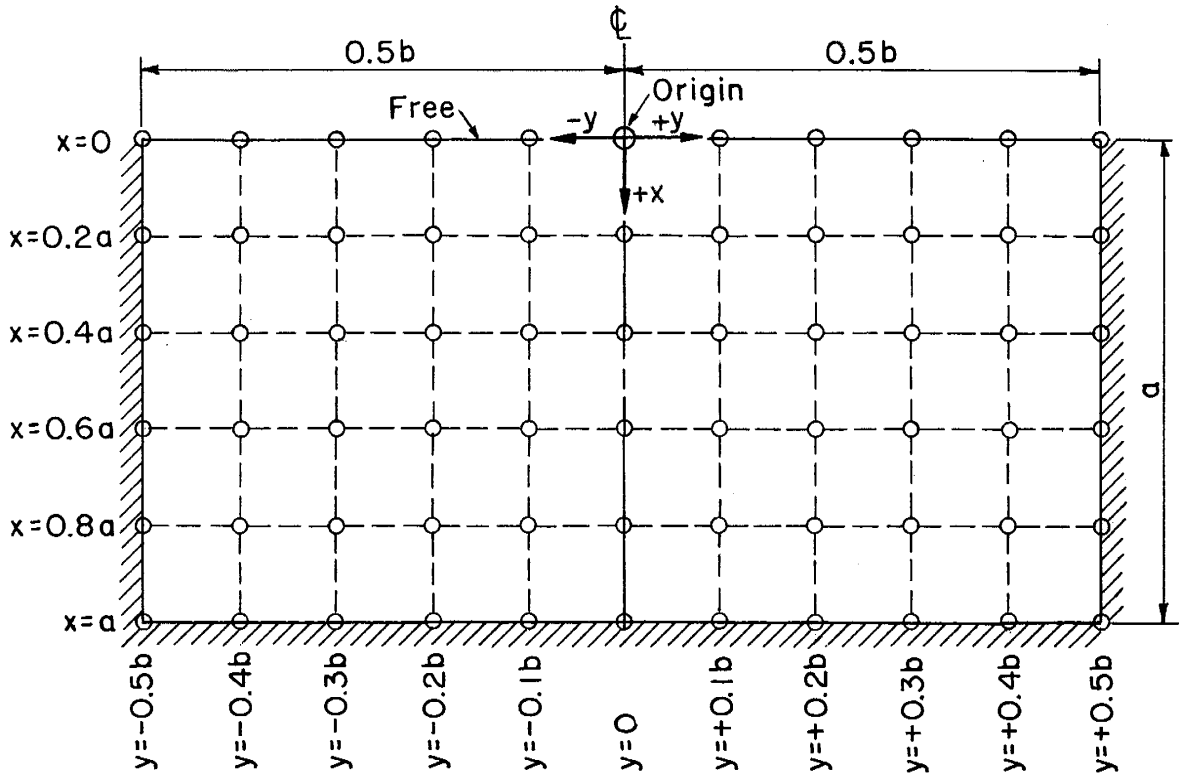
Caution: Use proper sign of fixed end moments when substituting them in the given equations.

Nomenclature:

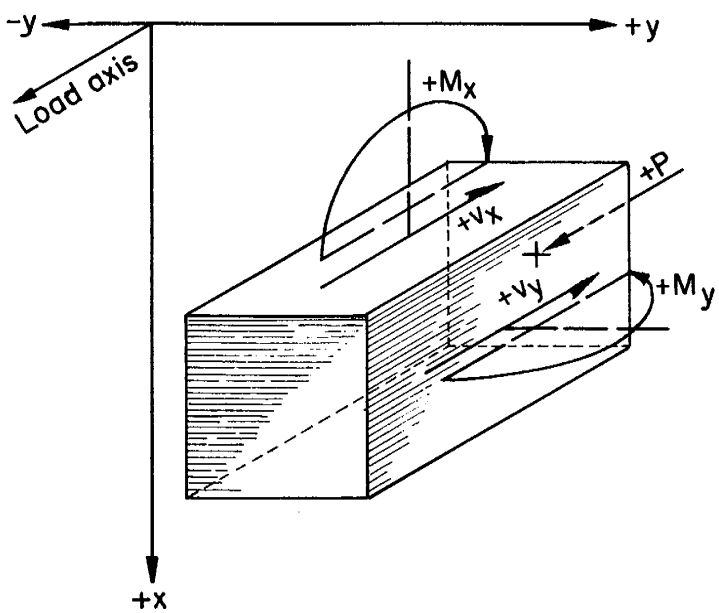
M_{ab} = Moment at end "a" of member "ab".
M_{ab}^F = Fixed end moment at end "a" of member "ab".
U_a^F = Algebraic sum of fixed end moments at joint a = unbalanced fixed end moment at a.
K_w = Stiffness of wall.
K_s = Stiffness of slab.

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING STANDARDS UNIT	STANDARD DWG. NO. ES-29 SHEET 1 OF 1 DATE 5-25-50

STRUCTURAL DESIGN : Cartesian grid point designation system, positive sign convention, symbols and nomenclature



CARTESIAN GRID POINT DESIGNATION SYSTEM



POSITIVE SIGN CONVENTION

SYMBOLS AND NOMENCLATURE

a = vertical dimension of fixed slab, ft

b = horizontal dimension of fixed slab, ft

M_x = vertical moment, lb ft/ft

M_y = horizontal moment, lb ft/ft

p = intensity of pressure, lbs/ft^2

v = shearing reactions per unit length acting normal to the plane of the slab, lbs/ft

w = weight, lbs/ft^3

x,y = rectangular coordinates in the plane of the slab

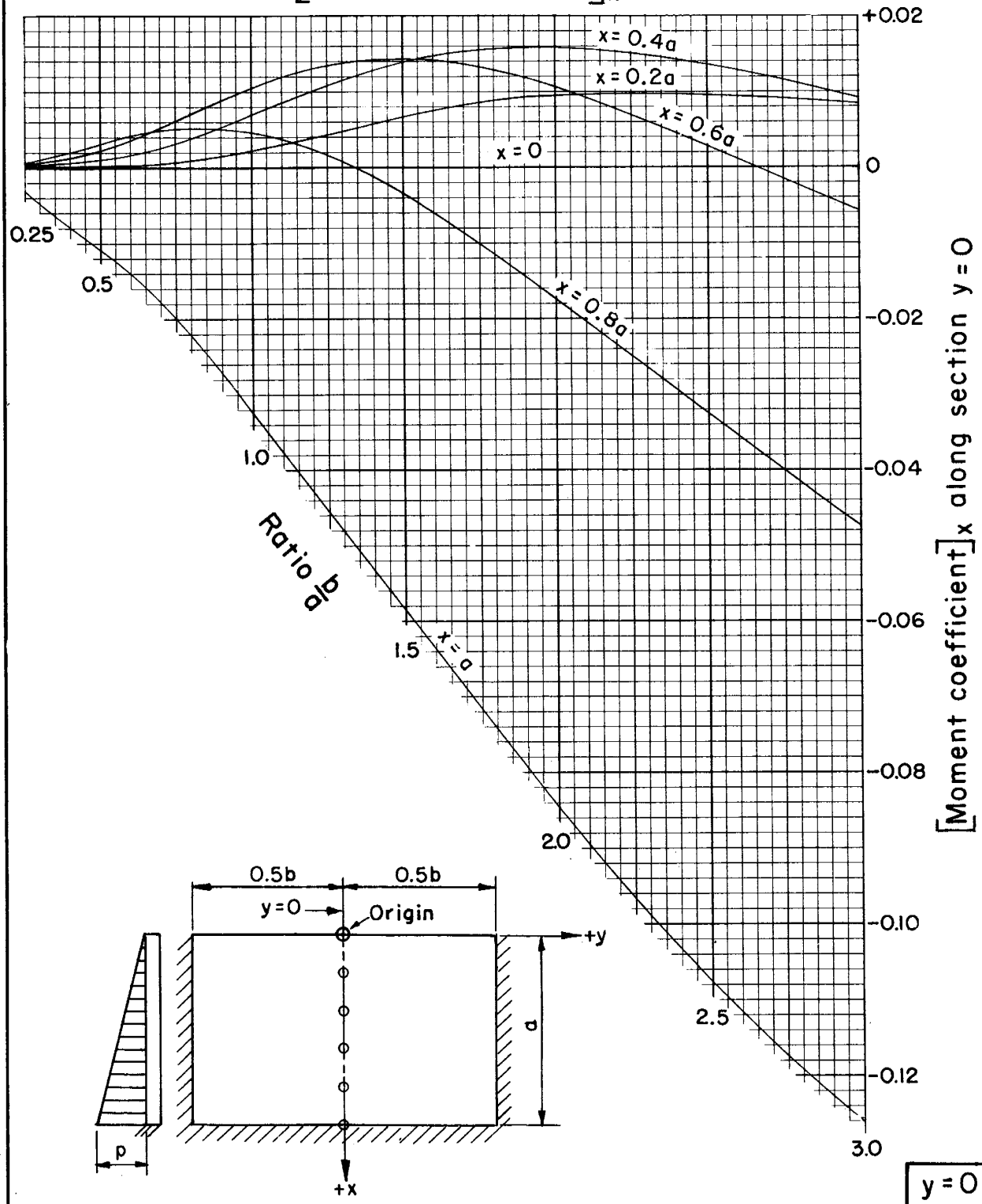
= fixed edge

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION-DESIGN SECTION	STANDARD DWG. NO. ES-104 SHEET 1 OF 85 DATE 4-13-56

STRUCTURAL DESIGN : Rectangular slabs with hydrostatic load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = 0$

Vertical moment determines tension in vertical steel

$$M_x = [\text{Moment coefficient}]_x \; pa^2$$

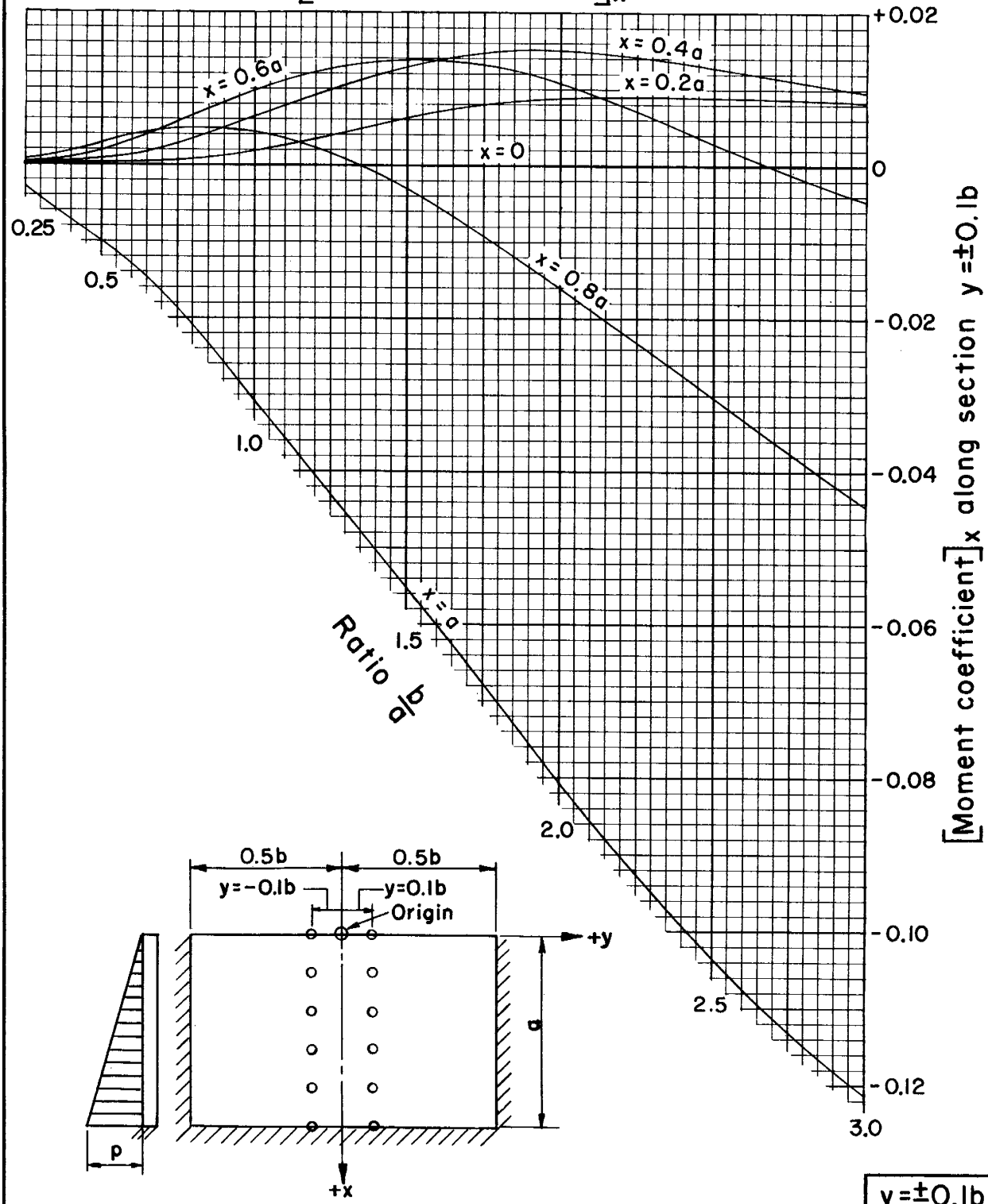


REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 2 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN : Rectangular slabs with hydrostatic load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.1b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \; pa^2$



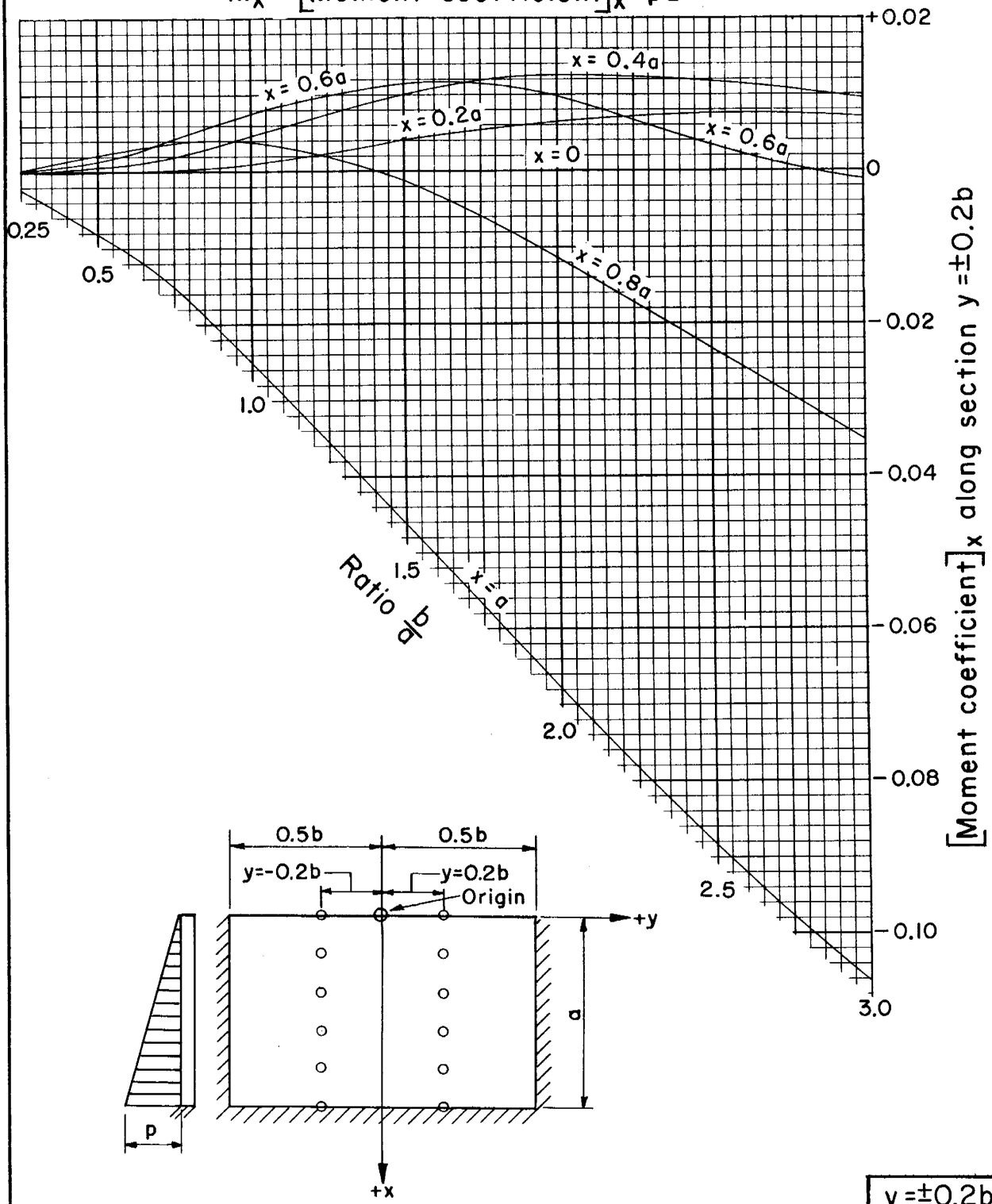
$y = \pm 0.1b$

REFERENCE		
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - DESIGN SECTION	STANDARD DWG. NO. ES-104 SHEET 3 OF 85 DATE 8-1-55

STRUCTURAL DESIGN : Rectangular slabs with hydrostatic load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.2b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \; pa^2$



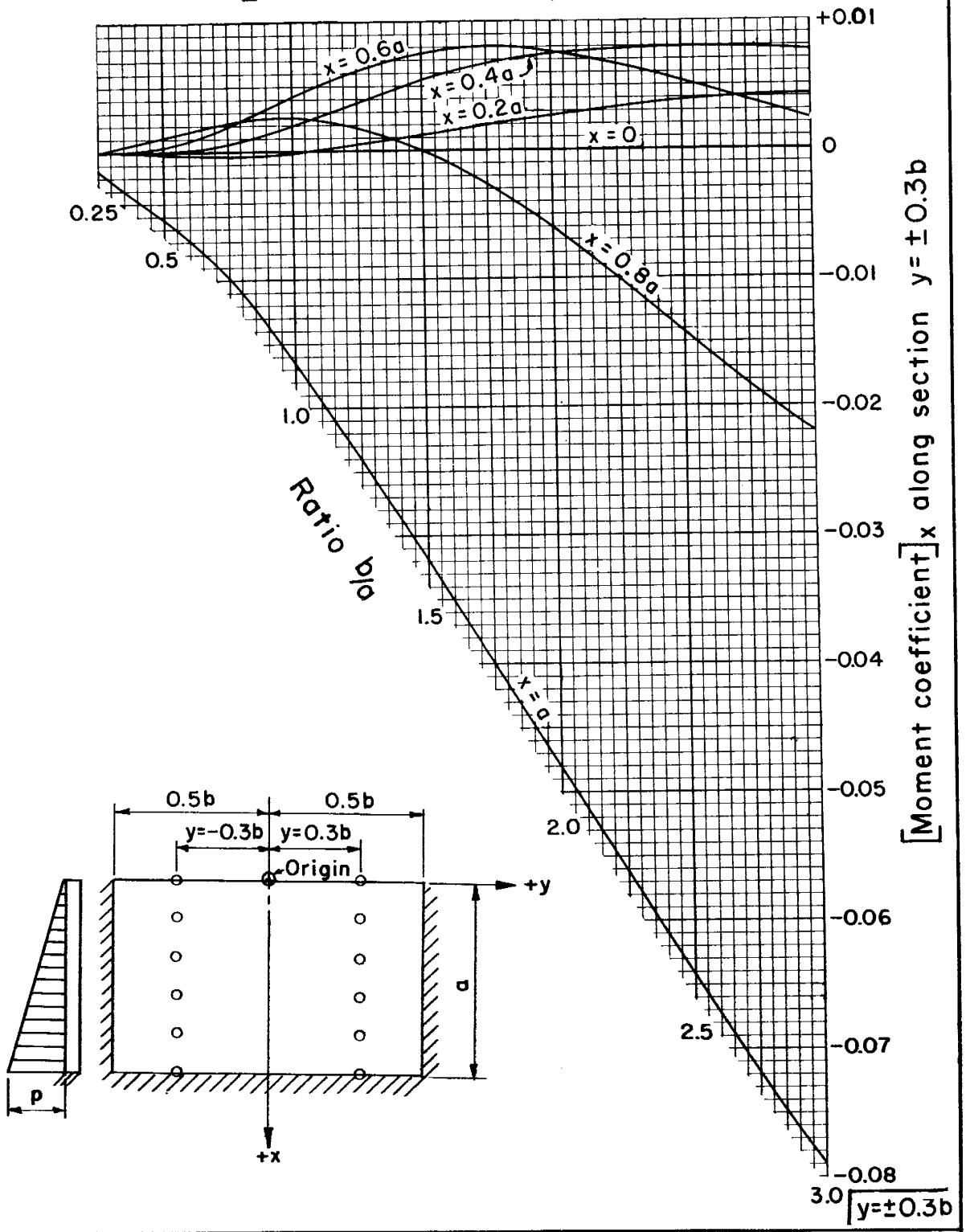
$y = \pm 0.2b$

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 4 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with hydrostatic load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.3b$

Vertical moment determines tension in vertical steel

$$M_x = [\text{Moment coefficient}]_x \; pa^2$$

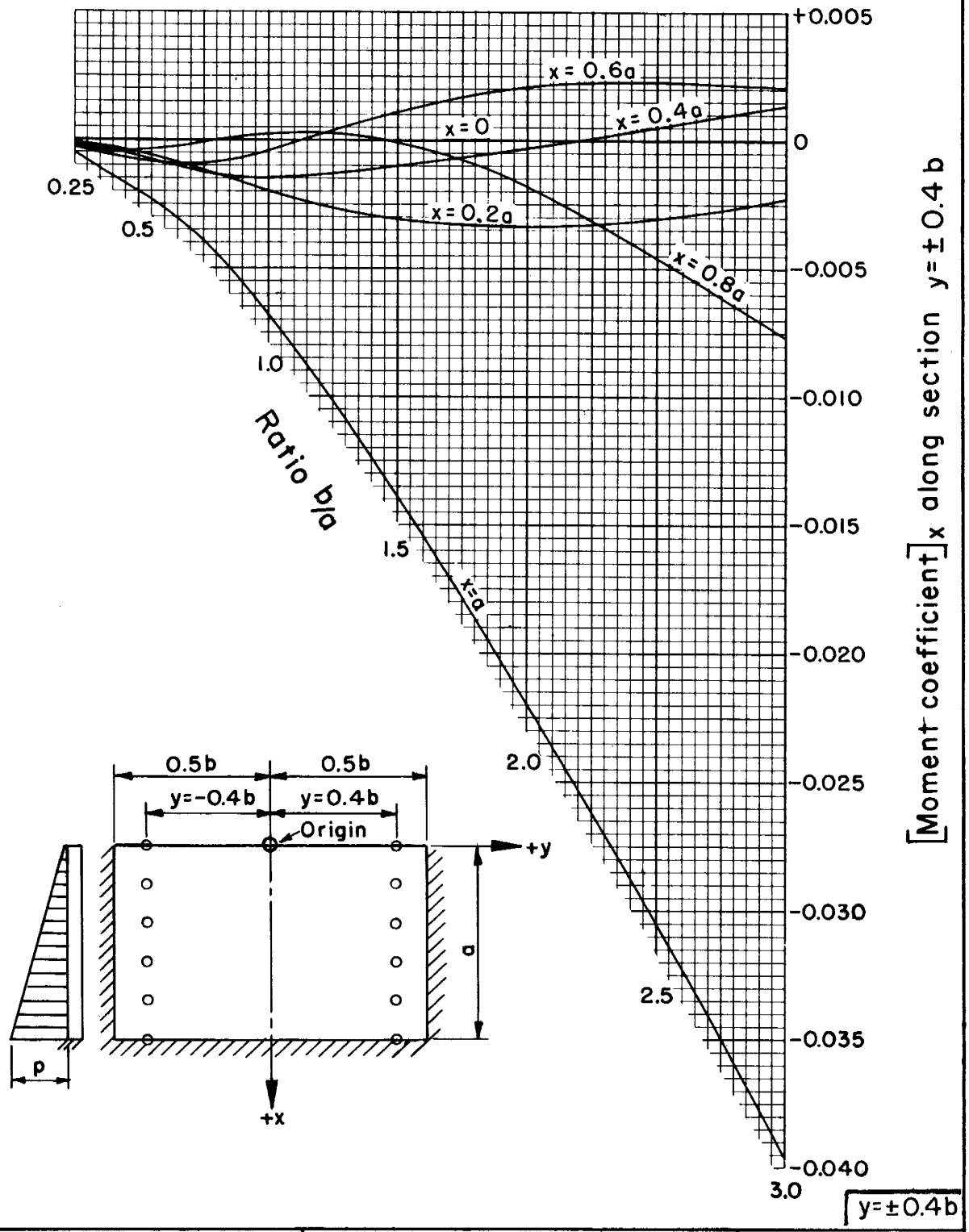


REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 5 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with hydrostatic load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.4b$

Vertical moment determines tension in vertical steel

$$M_x = [\text{Moment coefficient}]_x \; pa^2$$

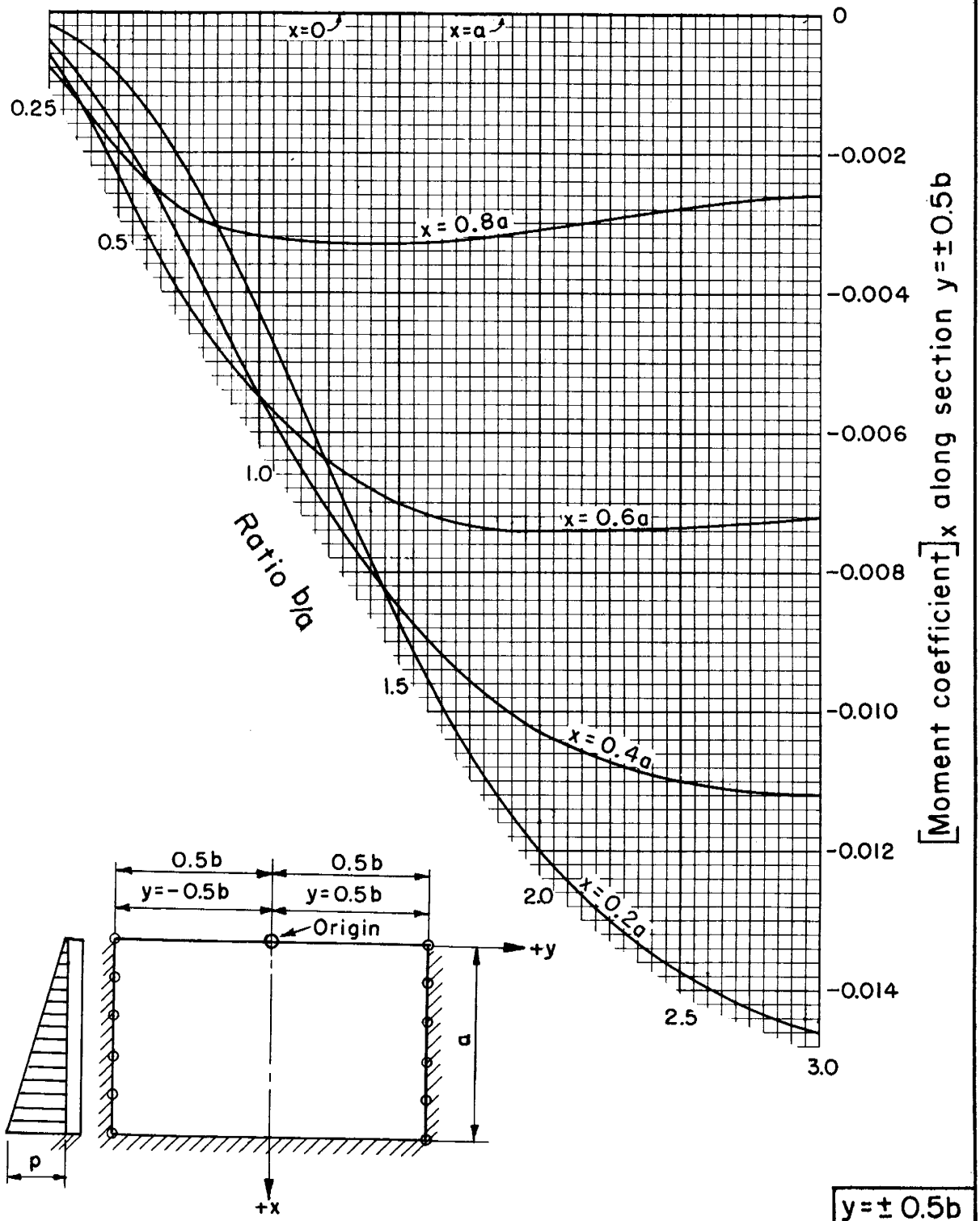


REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - DESIGN SECTION	STANDARD DWG. NO. ES-104 SHEET 6 OF 85 DATE 8-1-55
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954		

STRUCTURAL DESIGN : Rectangular slabs with hydrostatic load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.5b$

Vertical moment determines tension in vertical steel

$$M_x = [\text{Moment coefficient}]_x \; pa^2$$



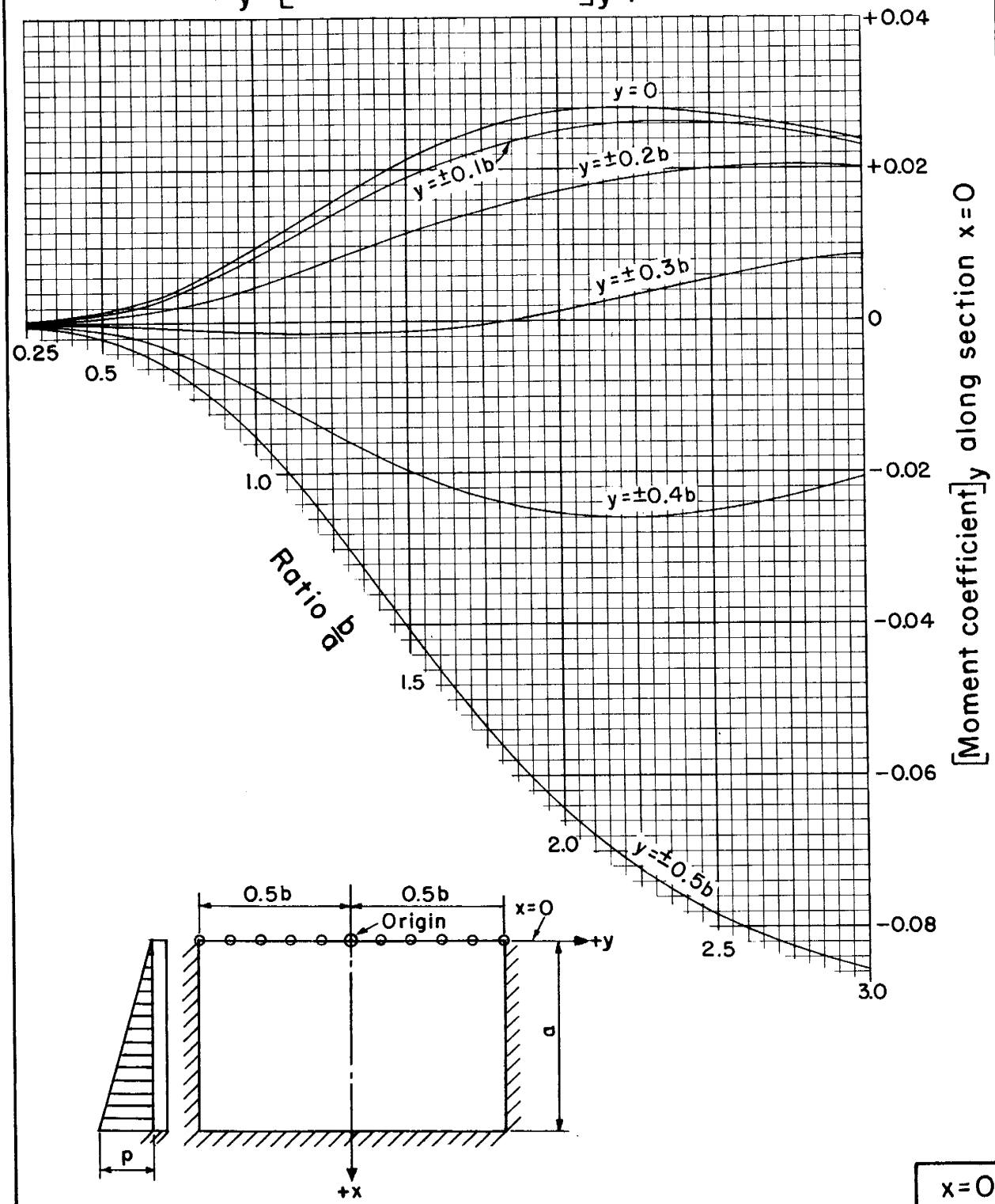
$y = \pm 0.5b$

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 7 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with hydrostatic load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice. x=0

Horizontal moment determines tension in horizontal steel

$M_y = [\text{Moment coefficient}]_y \, pa^2$



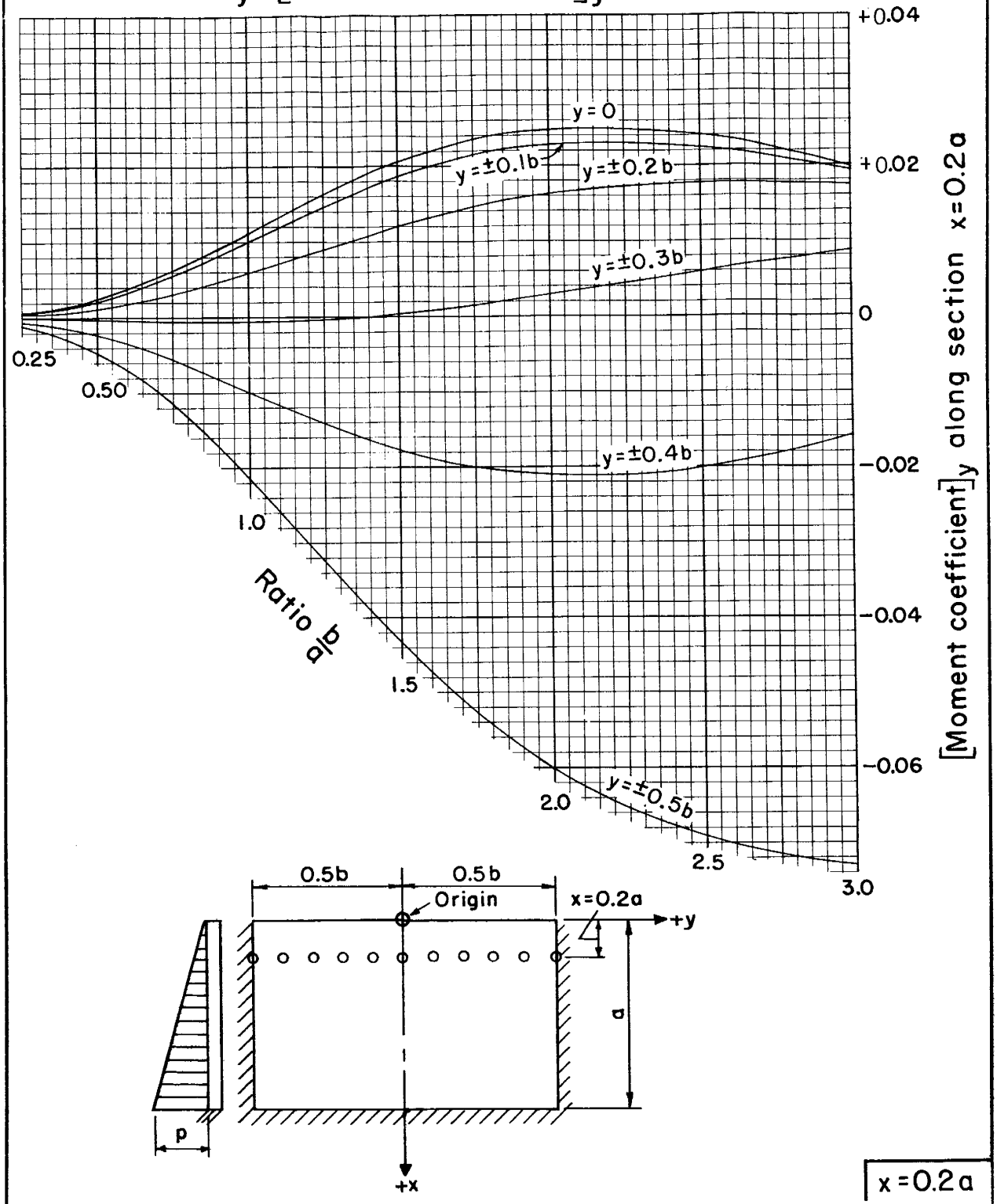
x=0

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 8 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with hydrostatic load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice. x = 0.2a

Horizontal moment determines tension in horizontal steel

$M_y = [\text{Moment coefficient}]_y \, pa^2$



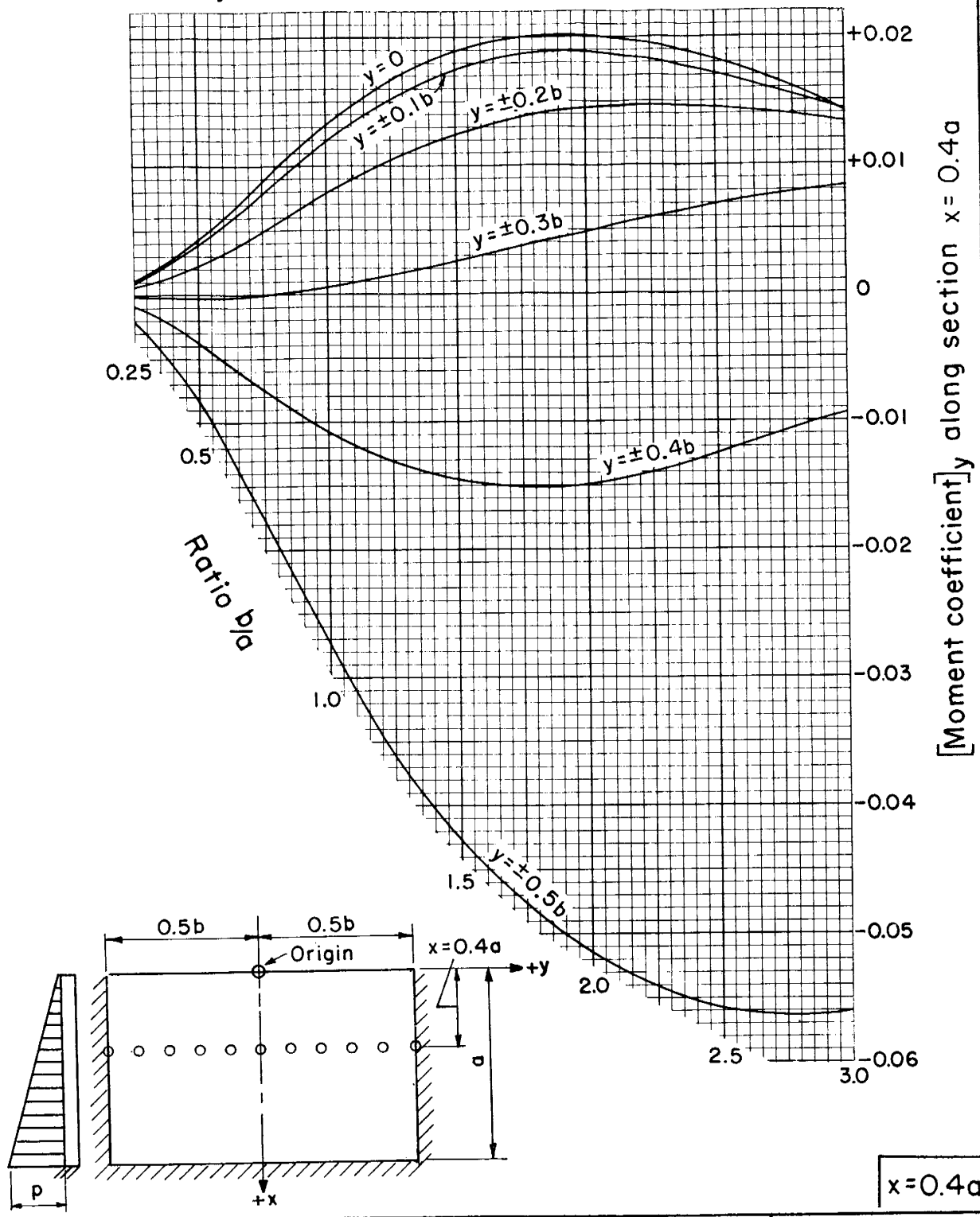
x = 0.2a

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 9 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with hydrostatic load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice. x= 0.4a

Horizontal moment determines tension in horizontal steel

$M_y = [\text{Moment coefficient}]_y \, pa^2$



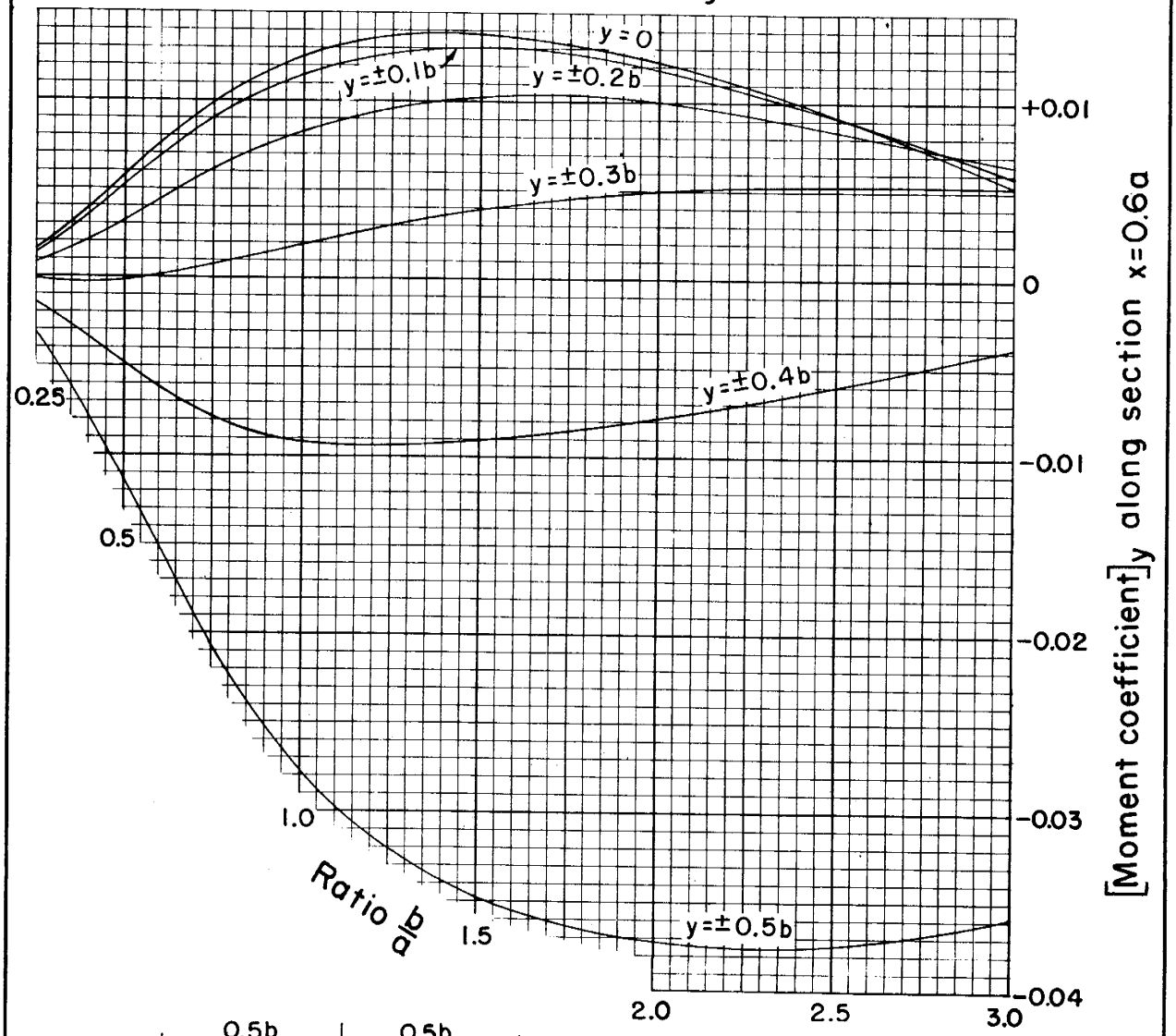
x=0.4a

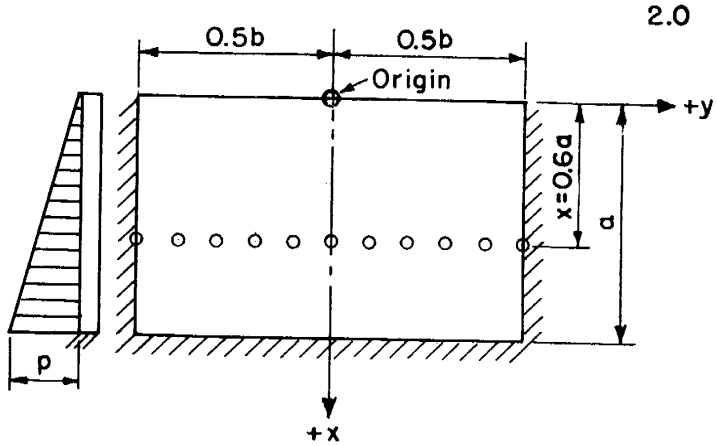
REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 10 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with hydrostatic load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice. x = 0.6a

Horizontal moment determines tension in horizontal steel

$M_y = [\text{Moment coefficient}]_y \, pa^2$





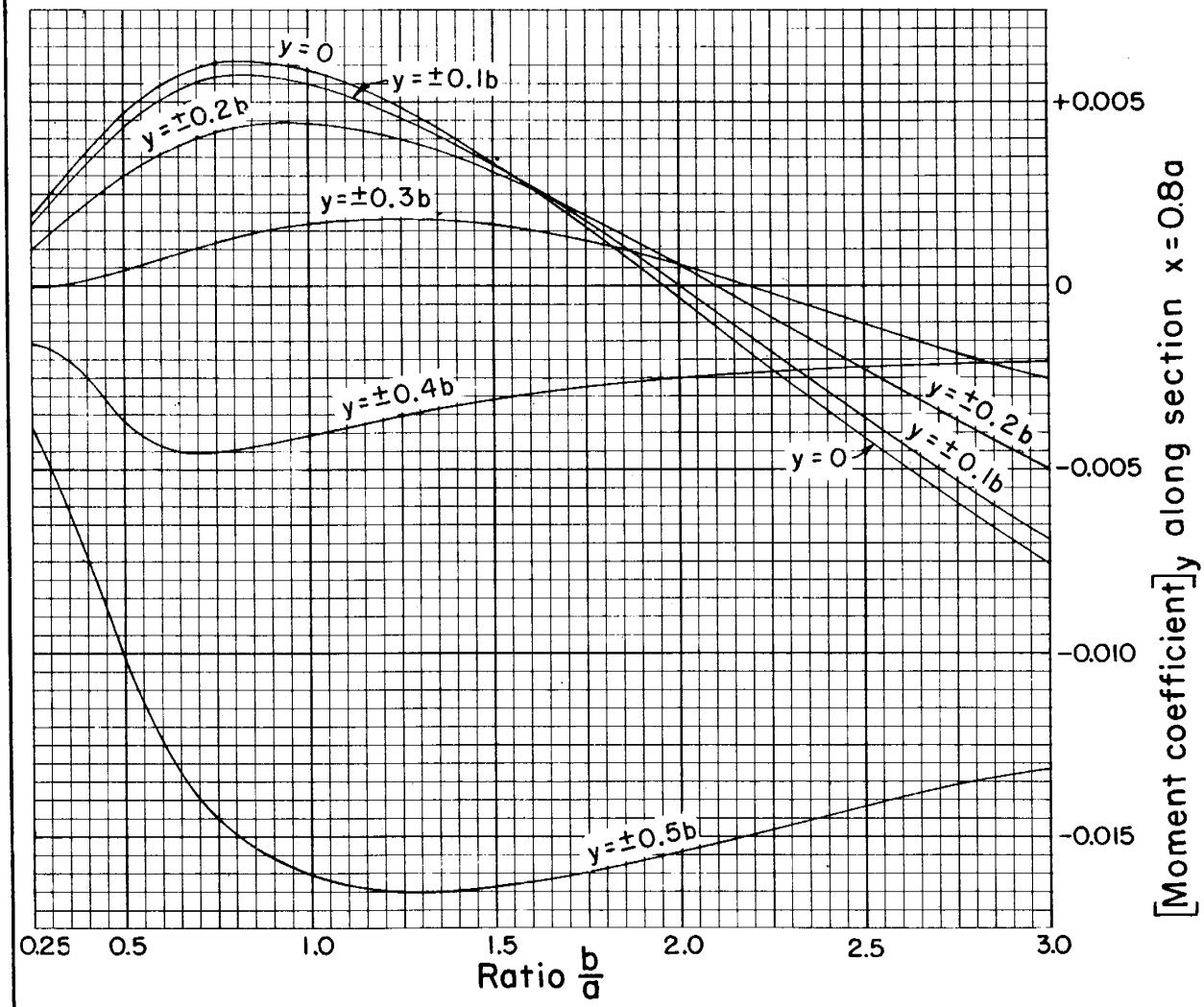
x = 0.6a

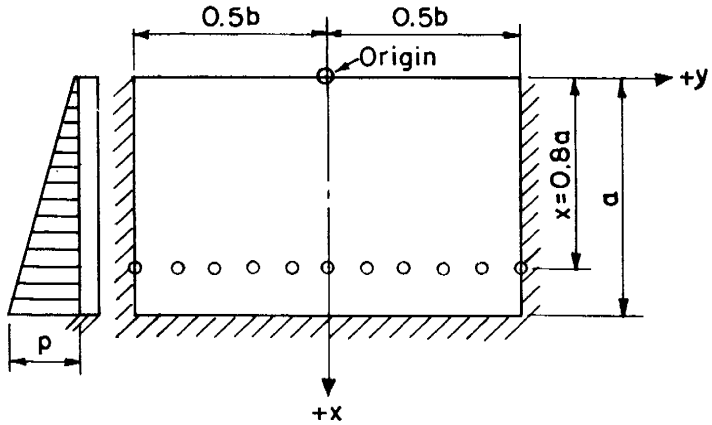
REFERENCE		
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - DESIGN SECTION	STANDARD DWG. NO. ES-104 SHEET 11 OF 85 DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with hydrostatic load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = 0.8a

Horizontal moment determines tension in horizontal steel

$M_y = [\text{Moment coefficient}]_y \, pa^2$





x=0.8a

REFERENCE
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954

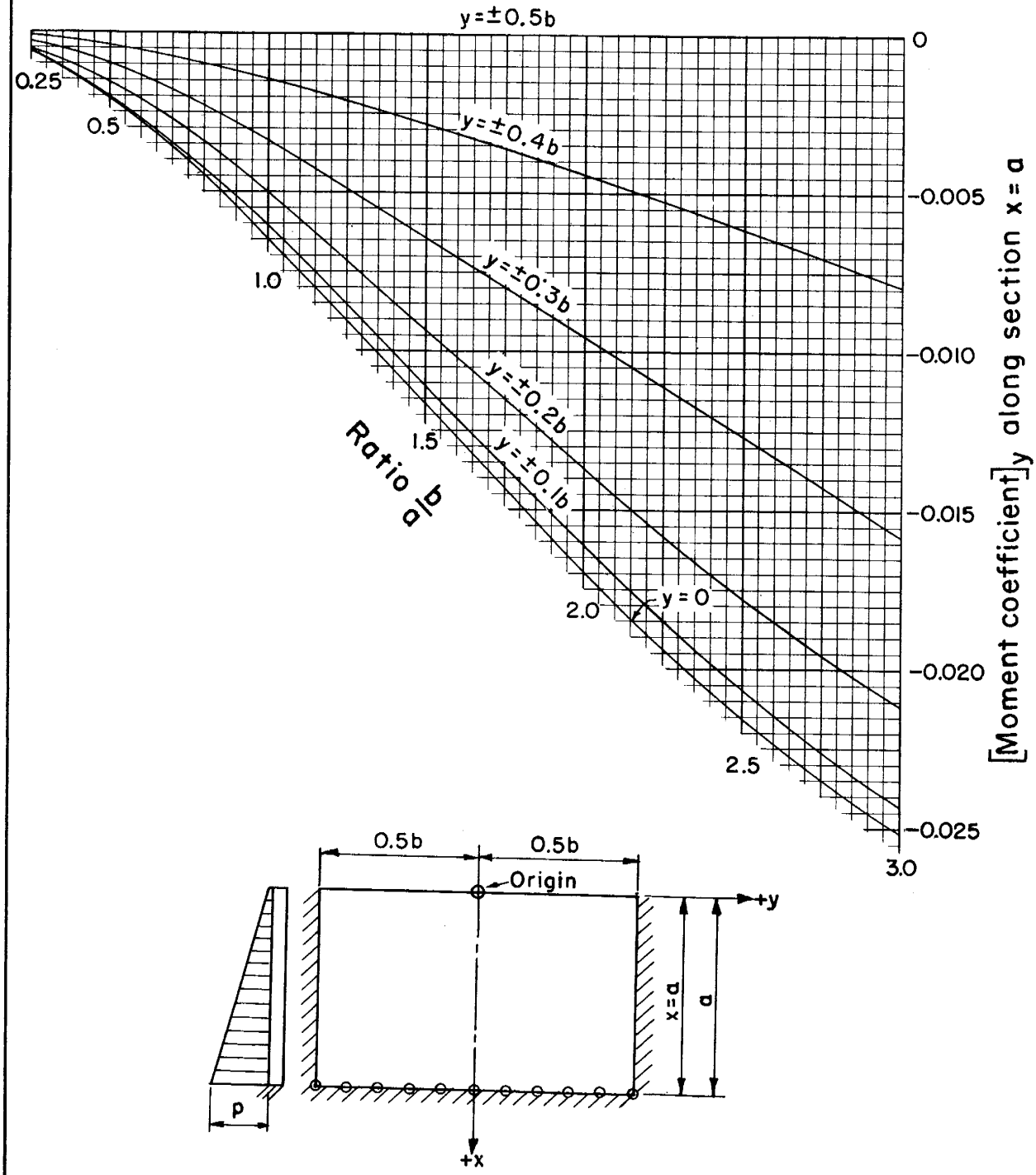
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES-104
SHEET 12 OF 85
DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with hydrostatic load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = a

Horizontal moment determines tension in horizontal steel

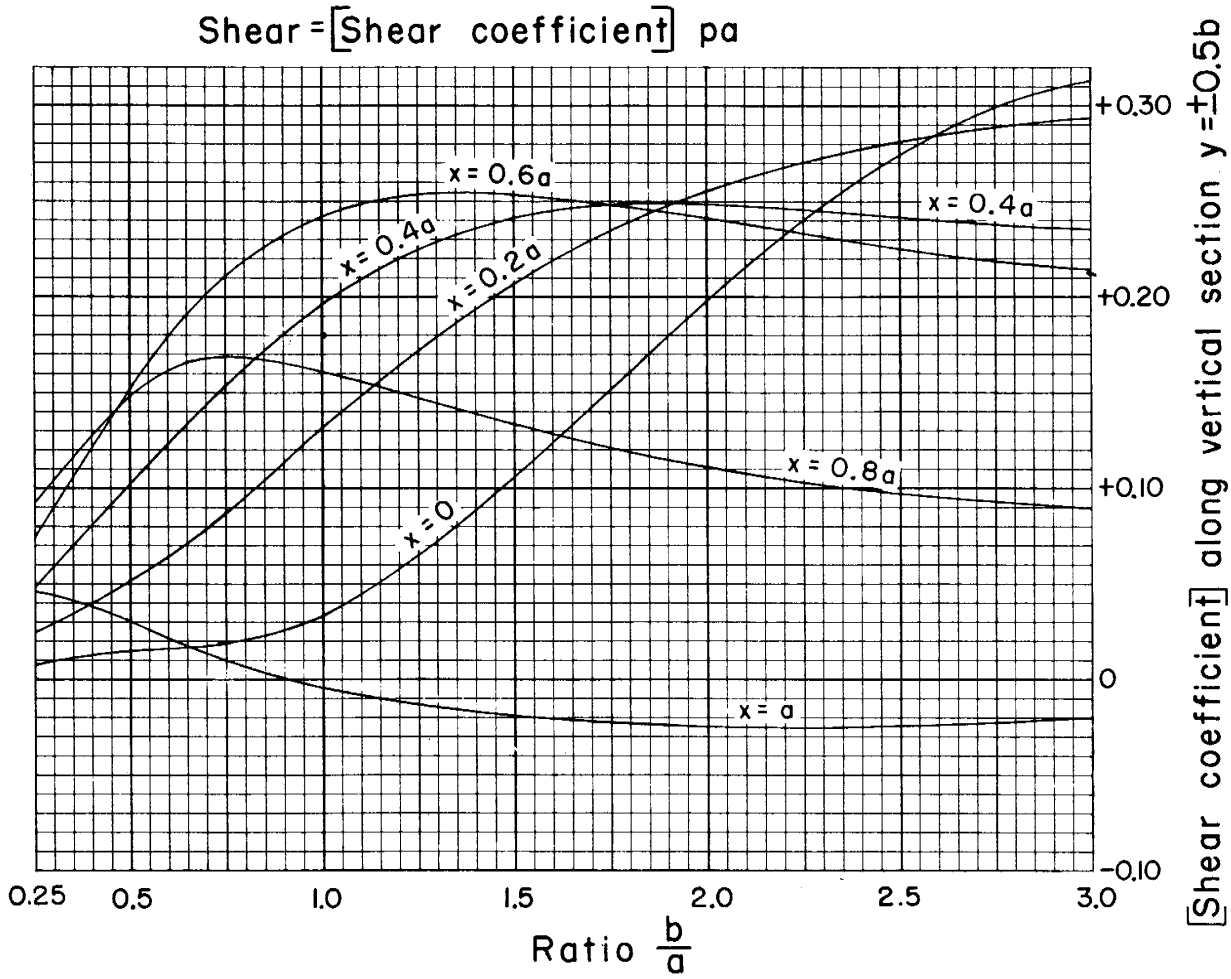
$$M_y = [\text{Moment coefficient}]_y \, pa^2$$

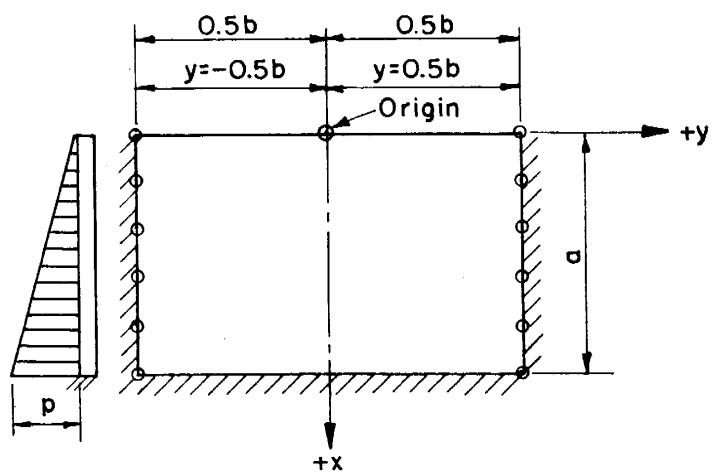


x = a

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 13 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with hydrostatic load; coefficients for shear at fifth points on fixed side edges $y = \pm 0.5b$



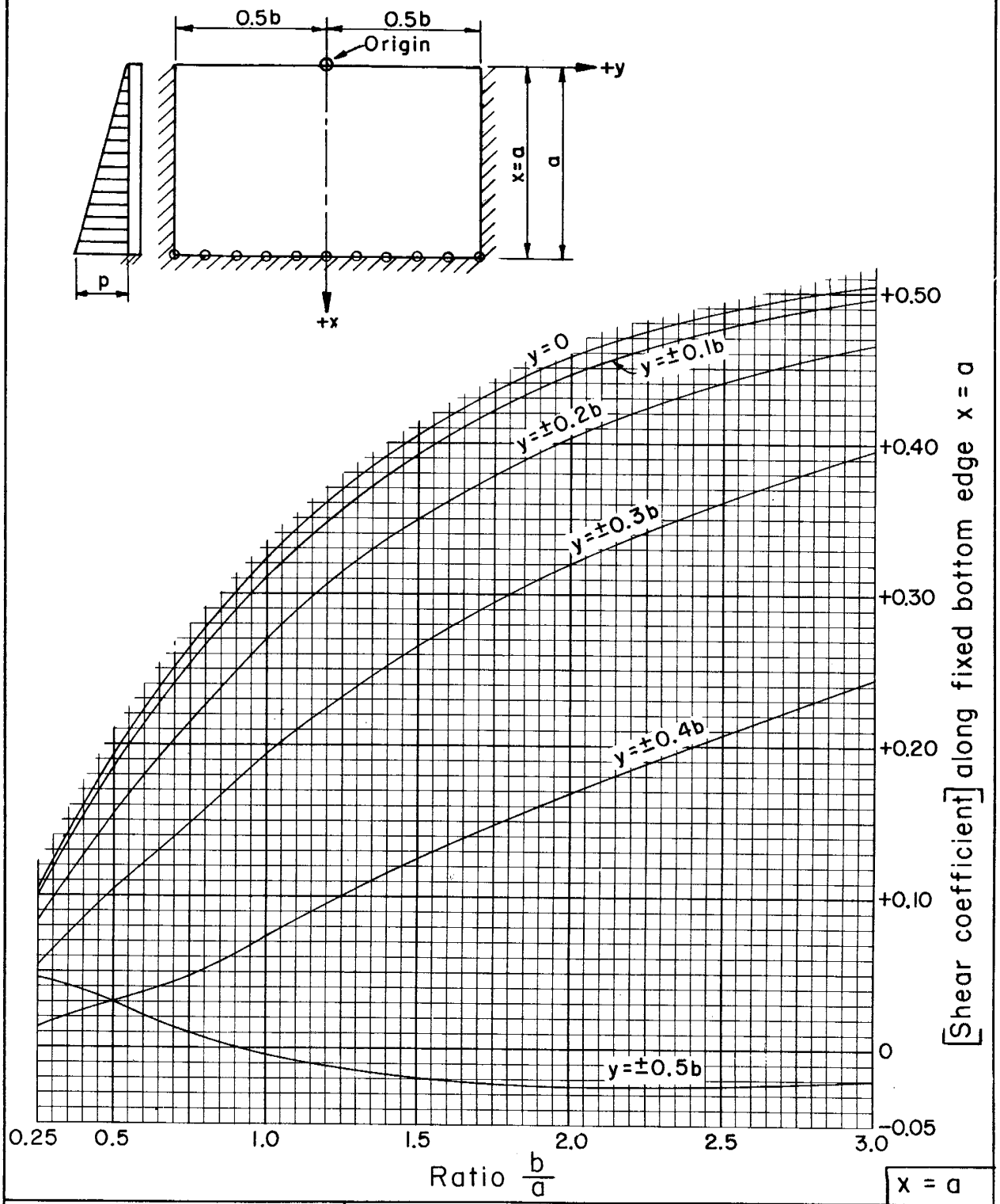


$y = \pm 0.5b$

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - DESIGN SECTION	STANDARD DWG. NO. ES-104 SHEET 14 OF 85 DATE 8-1-55
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954		

STRUCTURAL DESIGN : Rectangular slabs with hydrostatic load; coefficients for shear at tenth points on fixed bottom edge x = a

Shear = [Shear coefficient] pa



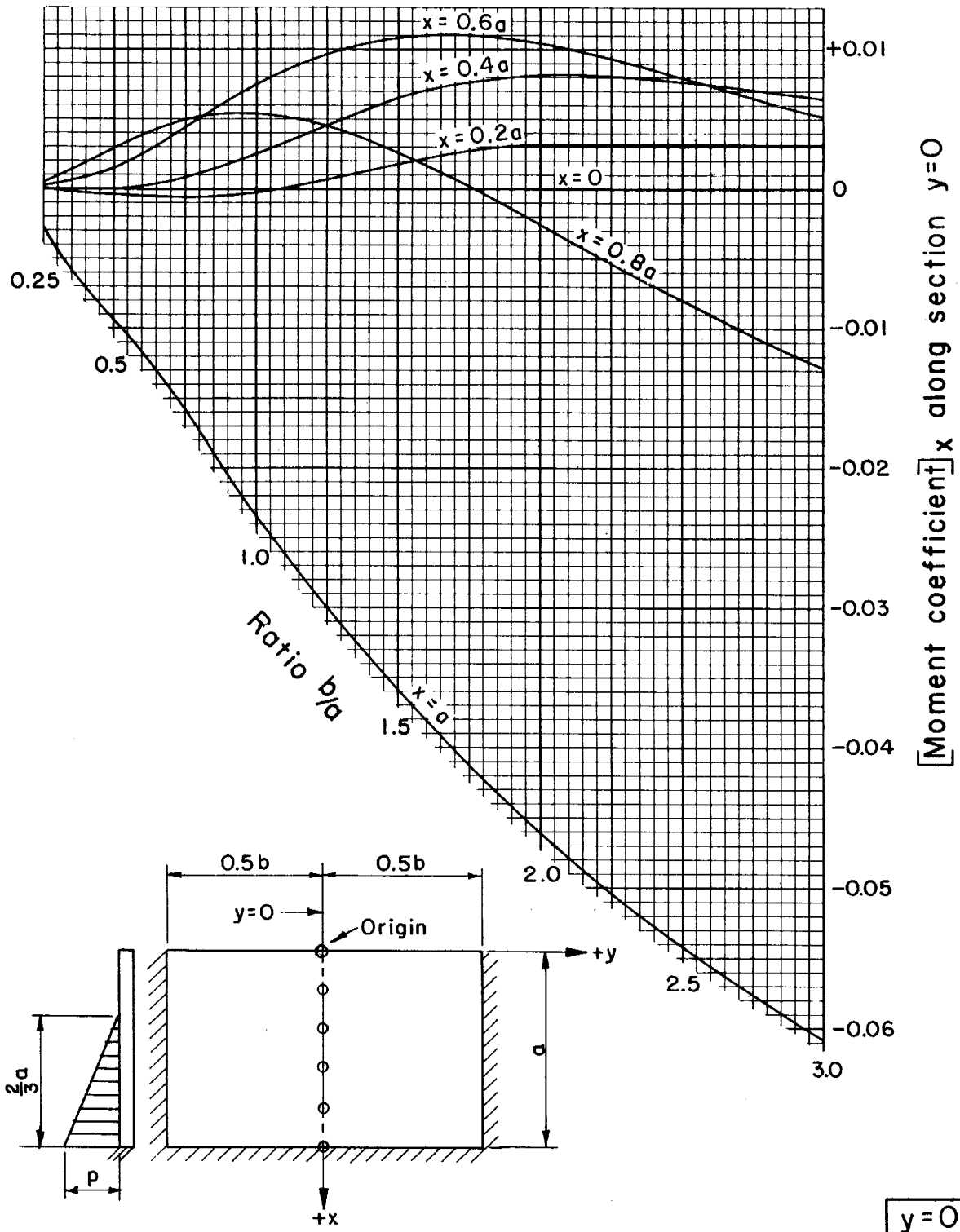
x = a

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 15 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with $^2/_3$ hydrostatic load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = 0$

Vertical moment determines tension in vertical steel

$$M_x = [\text{Moment coefficient}]_x \, pa^2$$



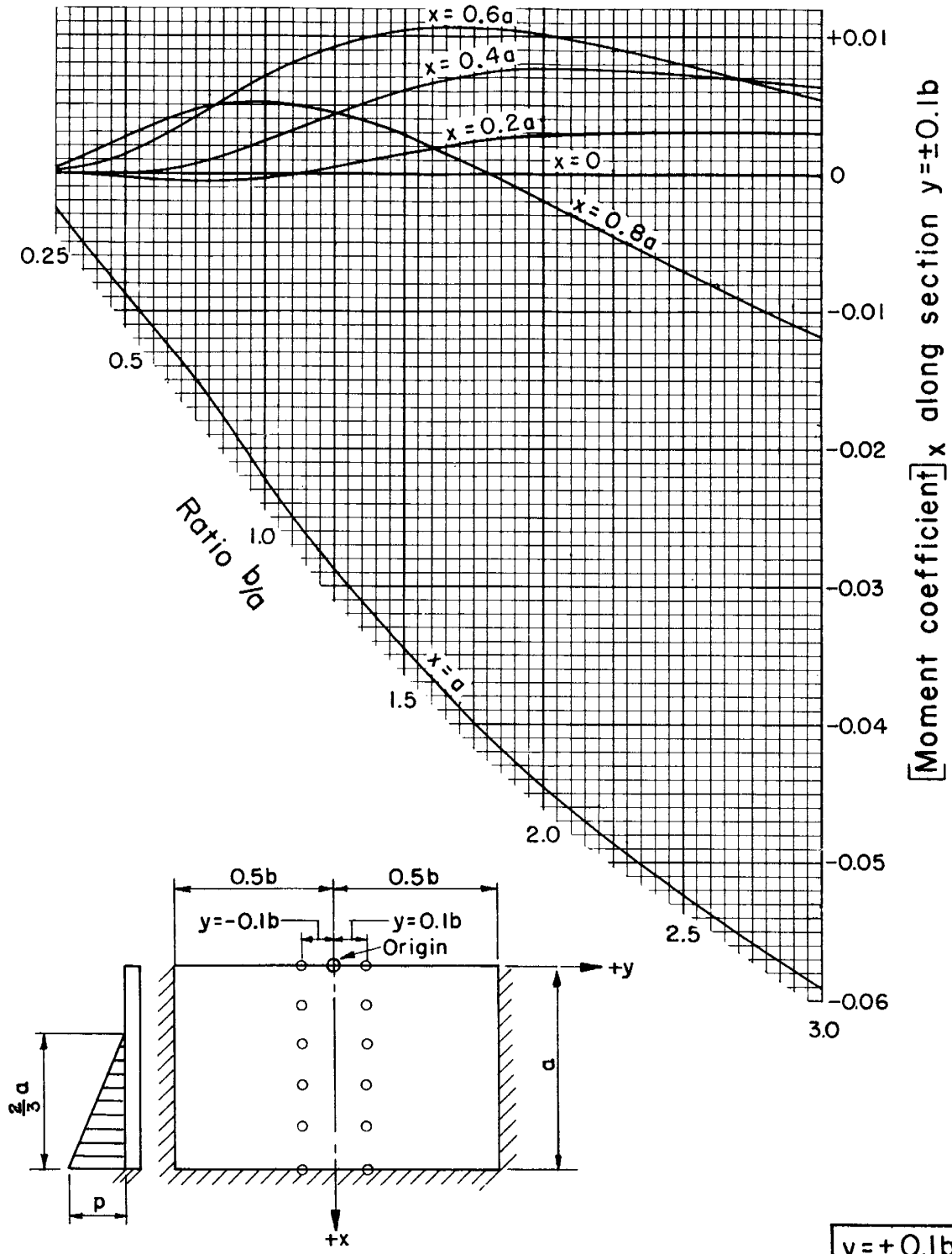
y = 0

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 16 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with $^2/_3$ hydrostatic load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.1b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \, pa^2$



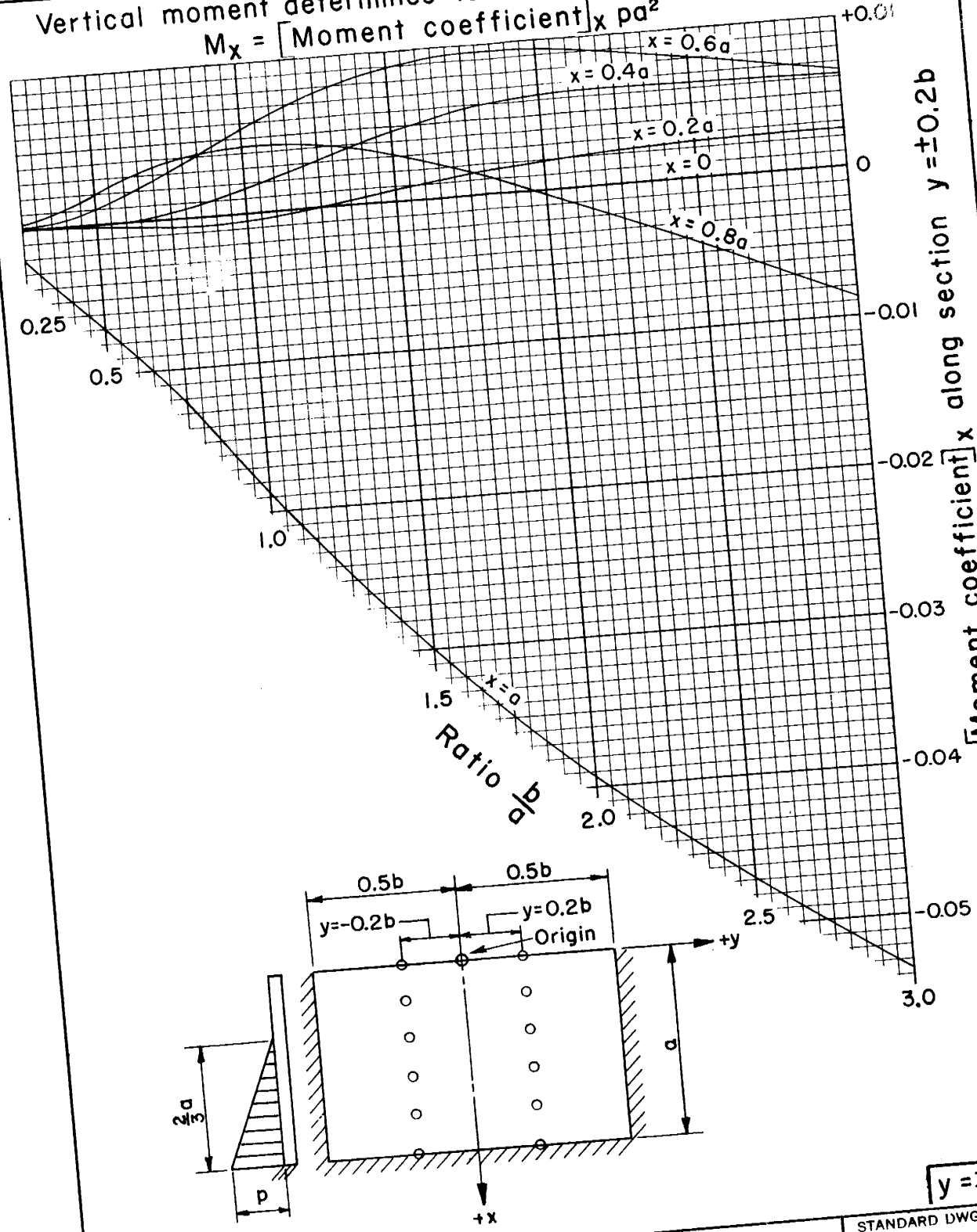
$y = \pm 0.1b$

REFERENCE		
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - DESIGN SECTION	STANDARD DWG. NO. ES-104 SHEET 17 OF 85 DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with 2/3 hydrostatic load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.2b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \, pa^2$



$y = \pm 0.2b$

REFERENCE

U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954

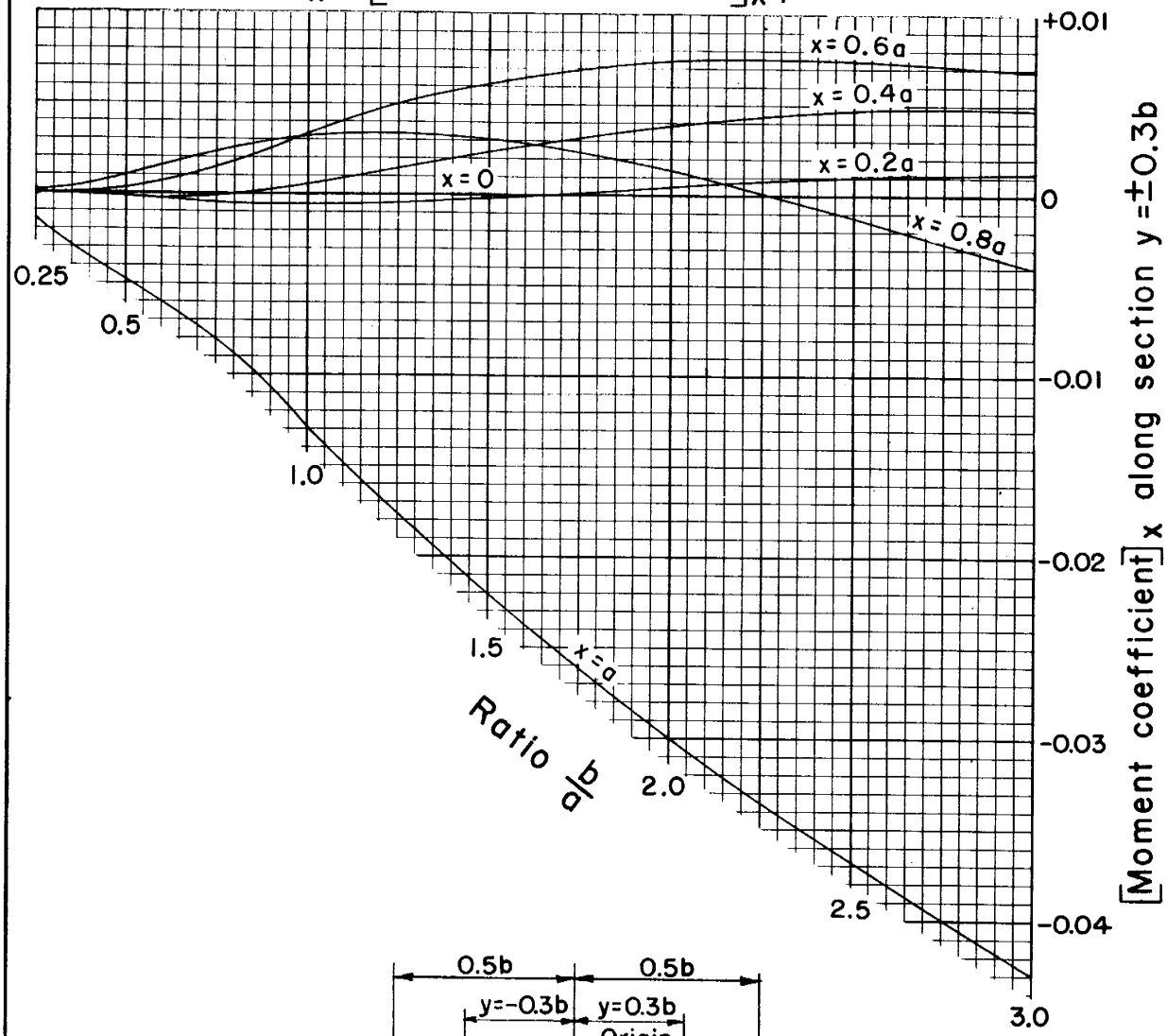
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

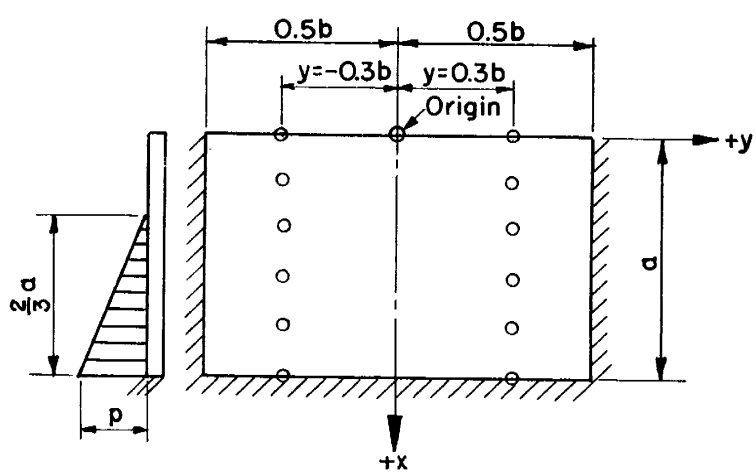
STANDARD DWG. NO.
ES-104
SHEET 18 OF 85
DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with $^2/_3$ hydrostatic load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.3b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \, pa^2$





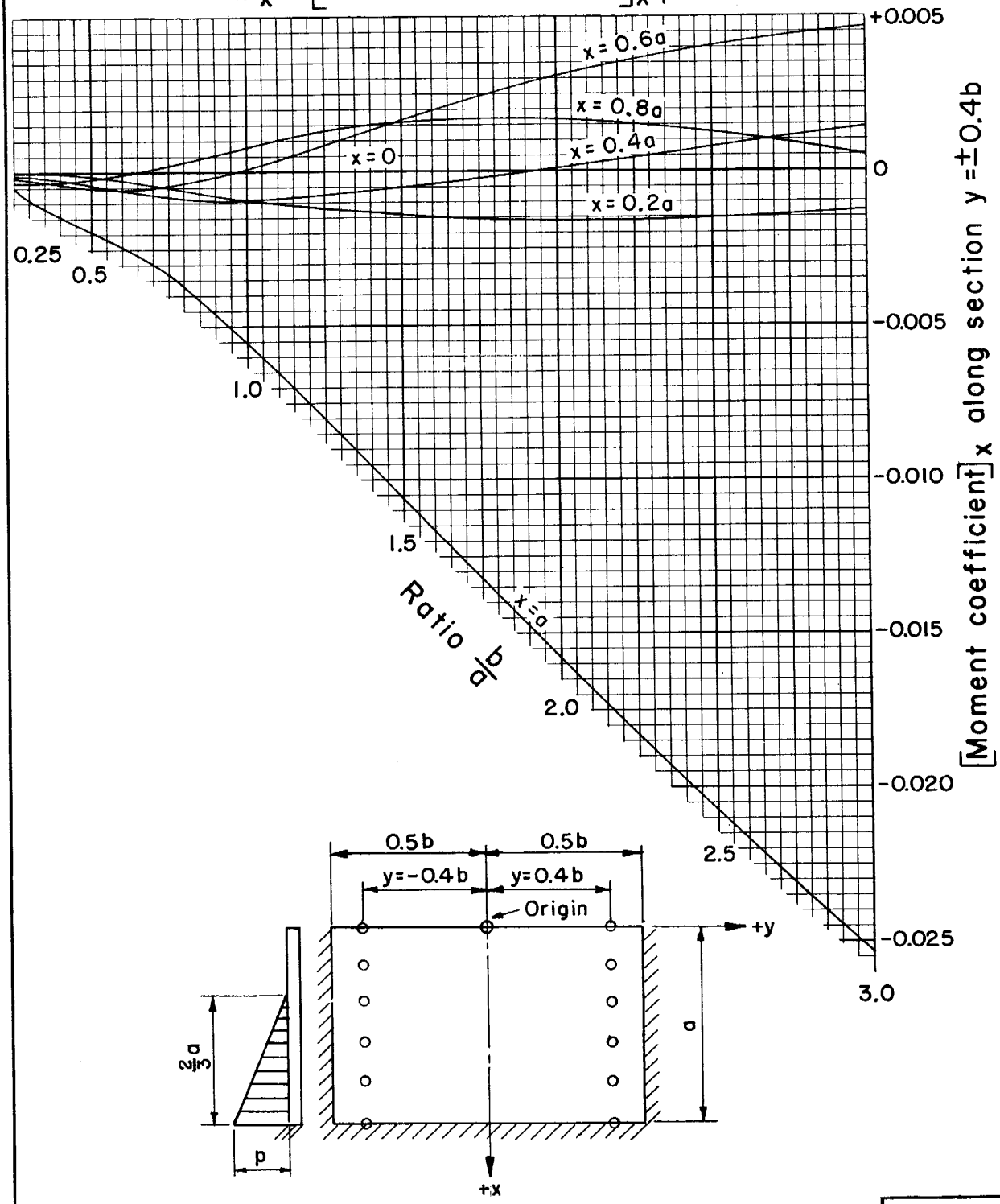
$y = \pm 0.3b$

REFERENCE		
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - DESIGN SECTION	STANDARD DWG. NO. ES-104 SHEET 19 OF 85 DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with $^2/_3$ hydrostatic load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.4b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \, pa^2$



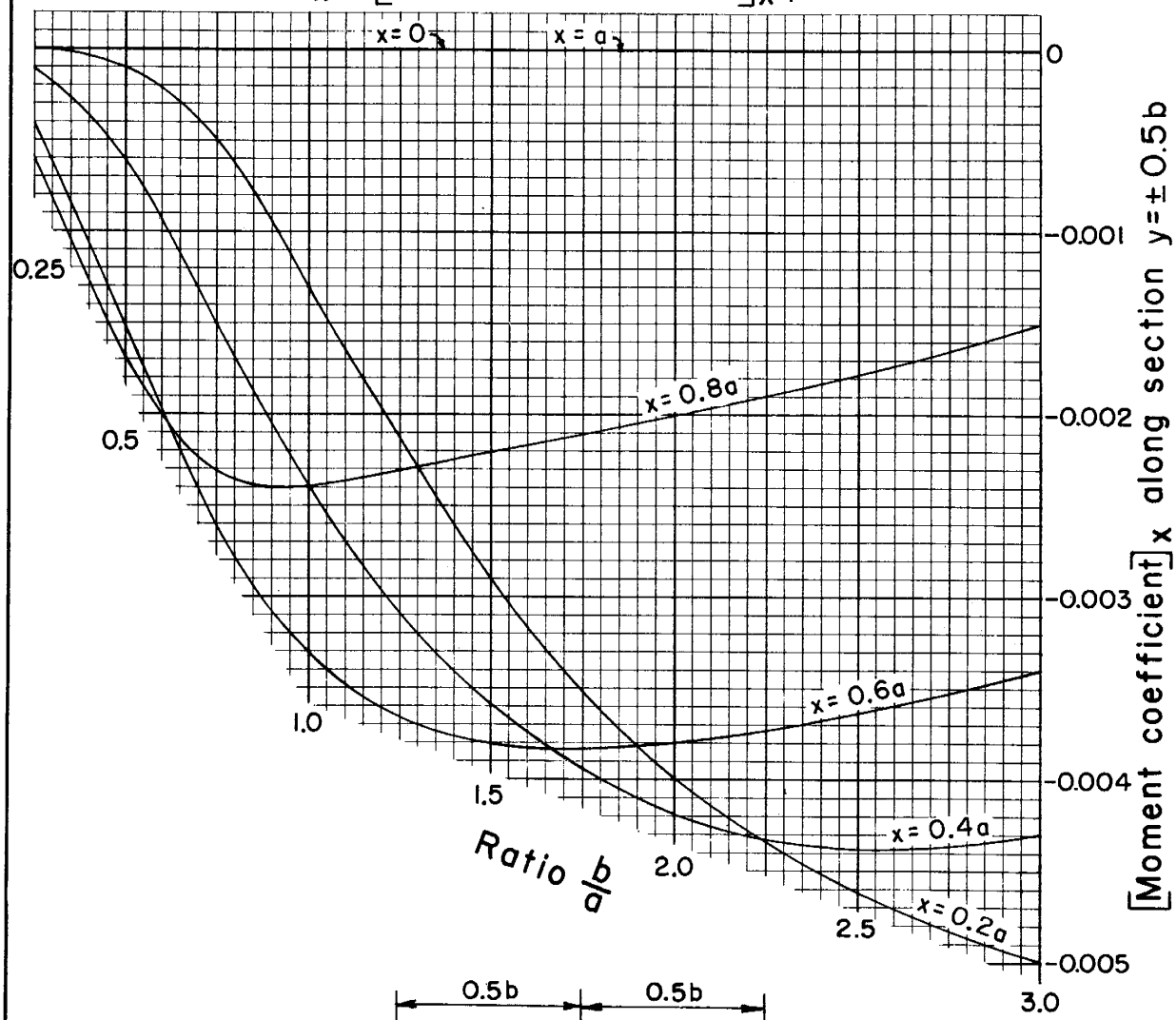
$y = \pm 0.4b$

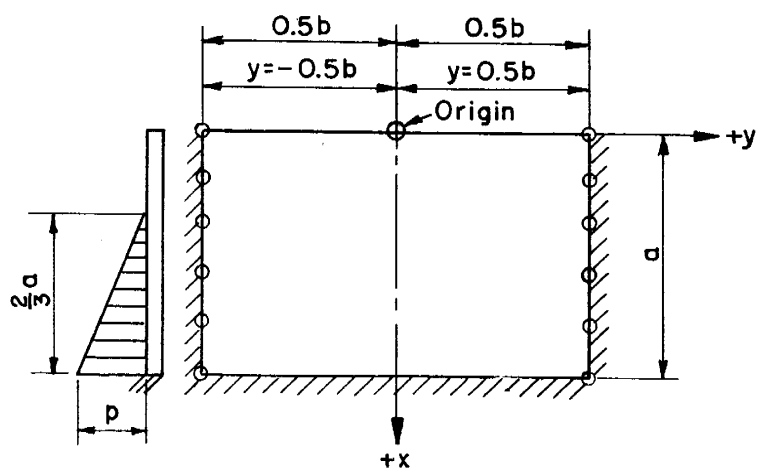
REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 20 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with $^2/_3$ hydrostatic load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.5b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \, pa^2$





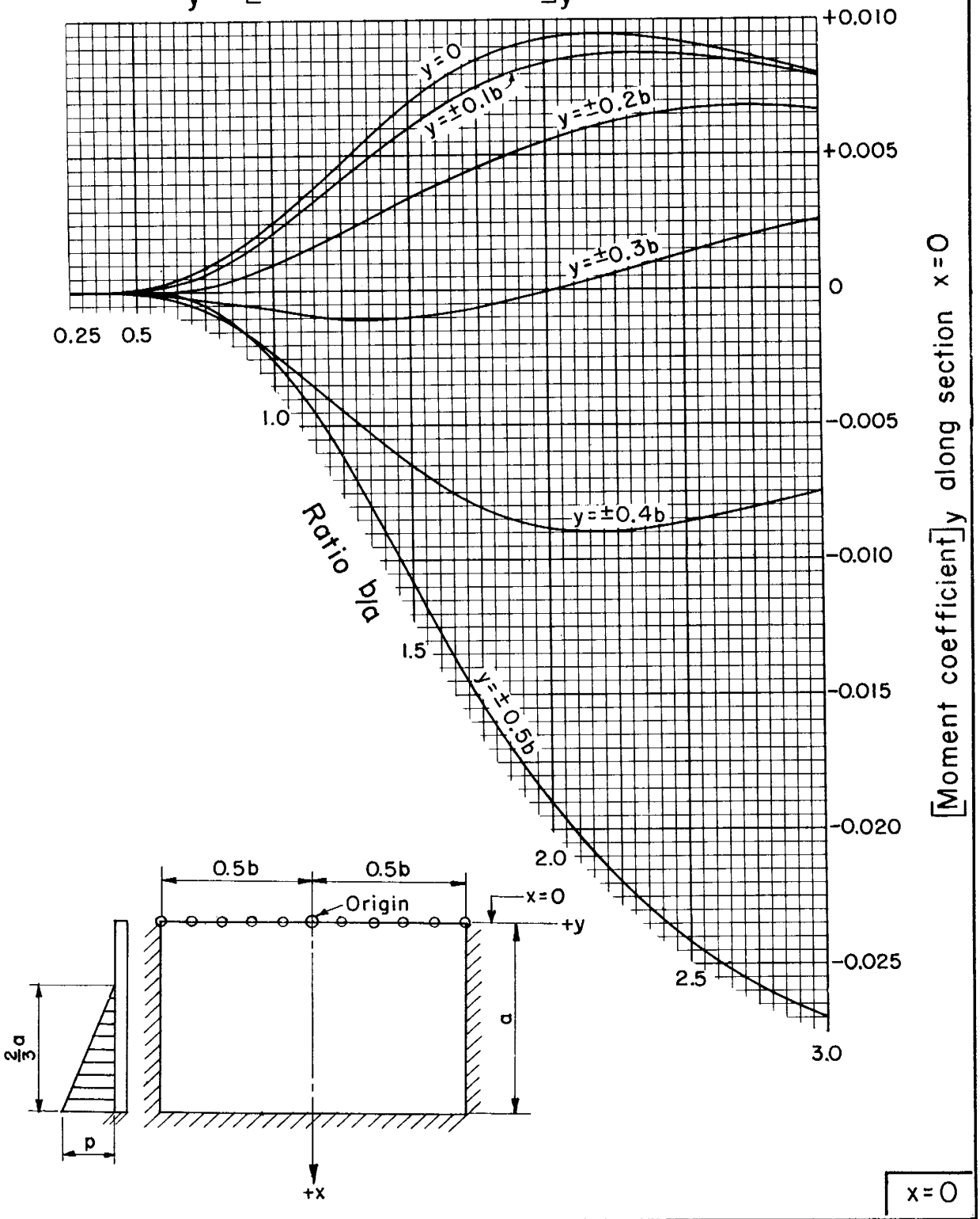
$y = \pm 0.5b$

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 21 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with $^2/_3$ hydrostatic load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = 0

Horizontal moment determines tension in horizontal steel

$$M_y = [\text{Moment coefficient}]_y \, pa^2$$



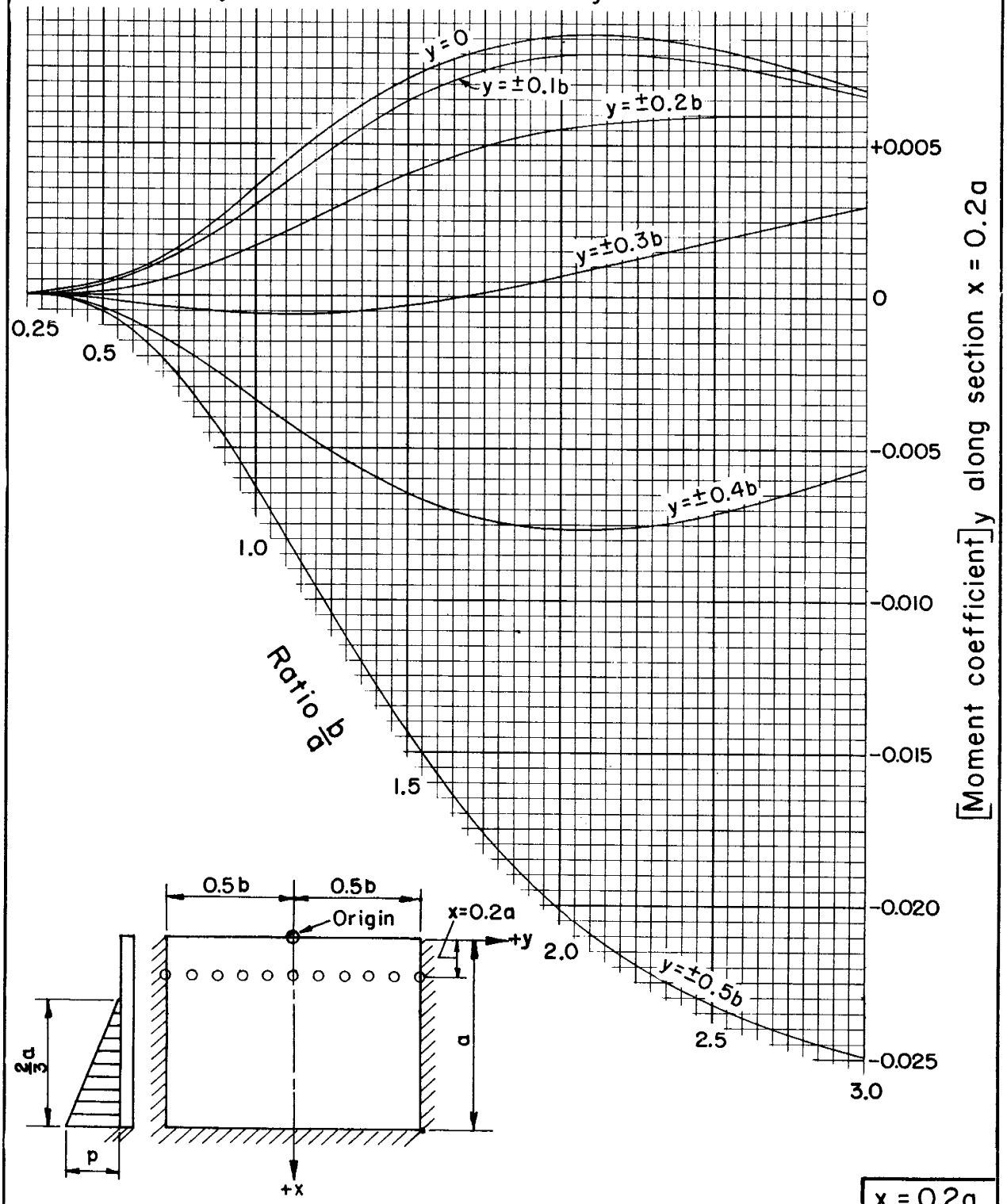
x = 0

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES- 104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 22 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with $^2/_3$ hydrostatic load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice $x = 0.2a$

Horizontal moment determines tension in horizontal steel

$$M_y = [\text{Moment coefficient}]_y \, pa^2$$



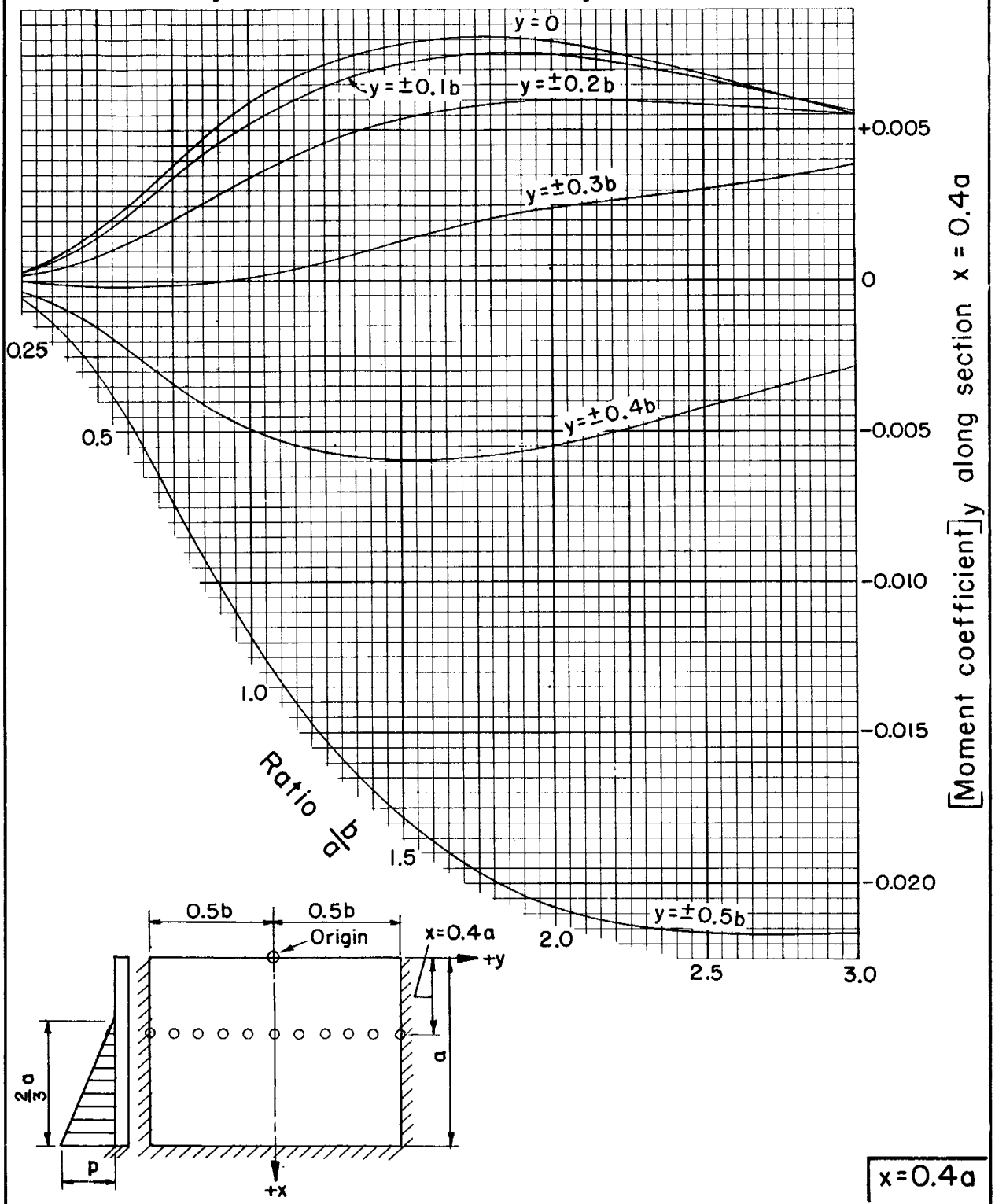
x = 0.2a

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES- 104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 23 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with $^2/_3$ hydrostatic load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice $x = 0.4a$

Horizontal moment determines tension in horizontal steel

$$M_y = [\text{Moment coefficient}]_y \, pa^2$$



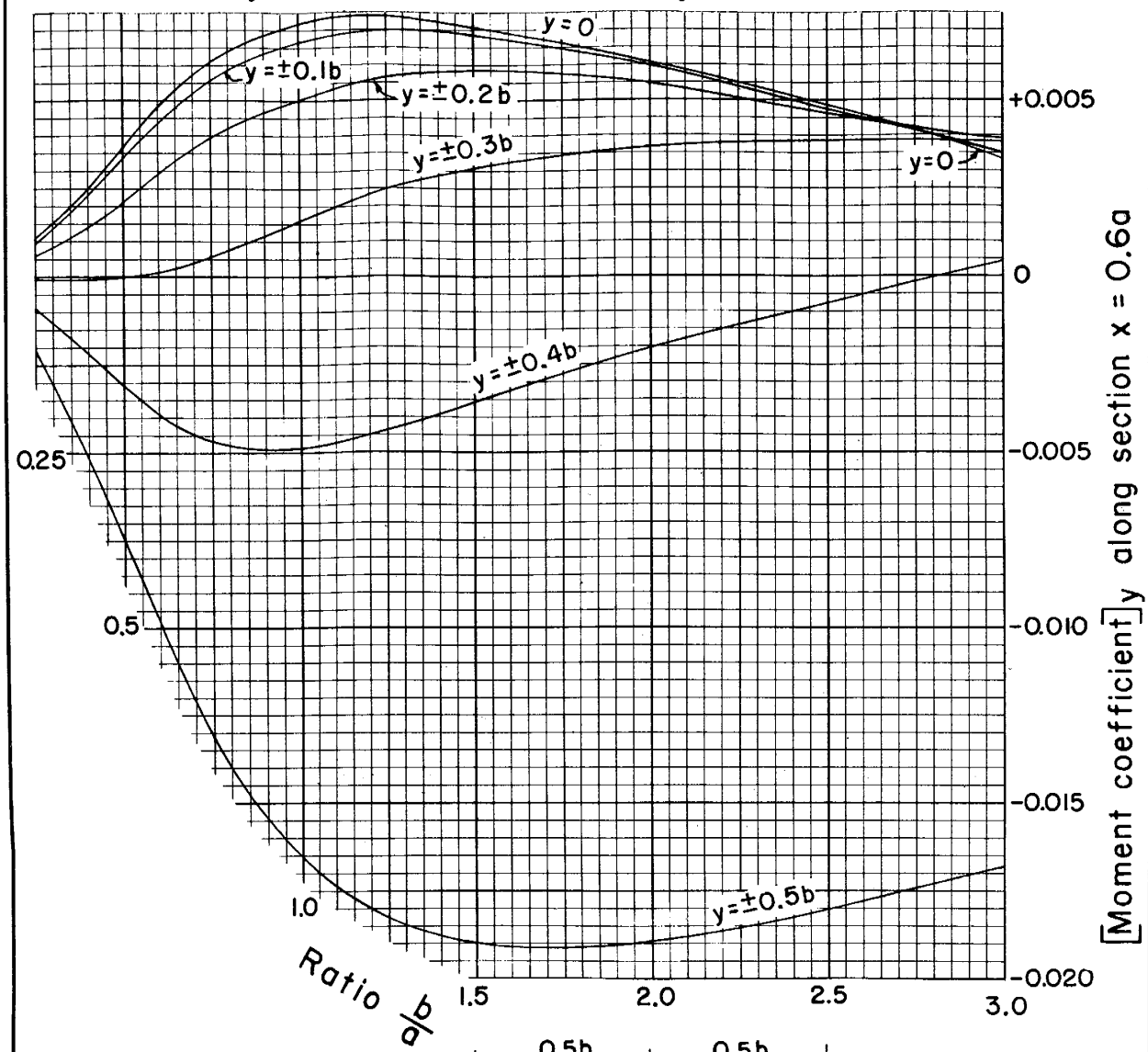
x=0.4a

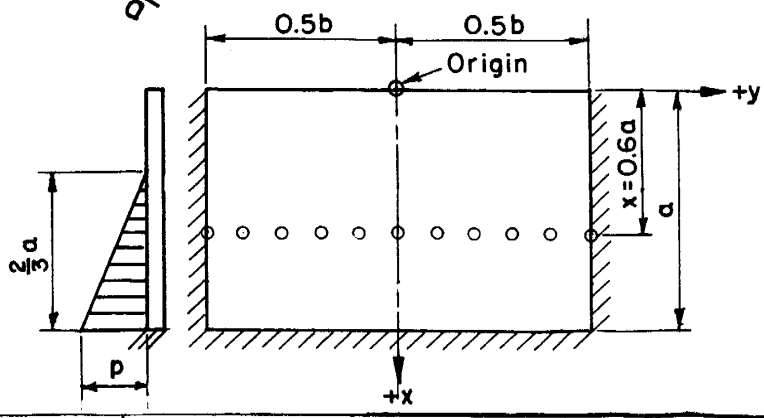
REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES- 104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 24 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with 2/3 hydrostatic load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = 0.6a

Horizontal moment determines tension in horizontal steel

M_y = [Moment coefficient]$_y$ pa^2





x = 0.6a

REFERENCE

U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954

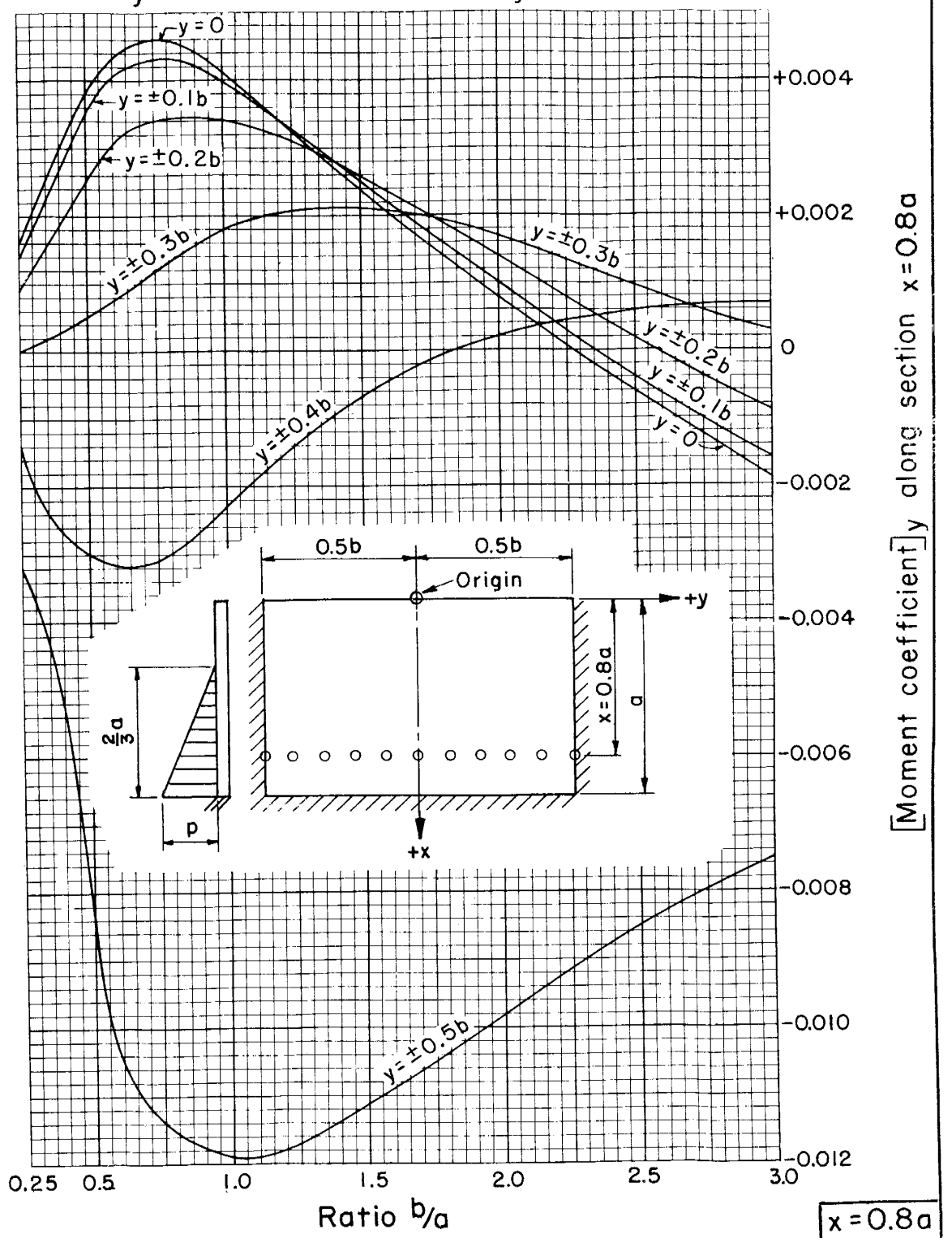
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 104
SHEET 25 OF 85
DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with 2/3 hydrostatic load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = 0.8a

Horizontal moment determines tension in horizontal steel

M_y = [Moment coefficient]$_y$ pa^2



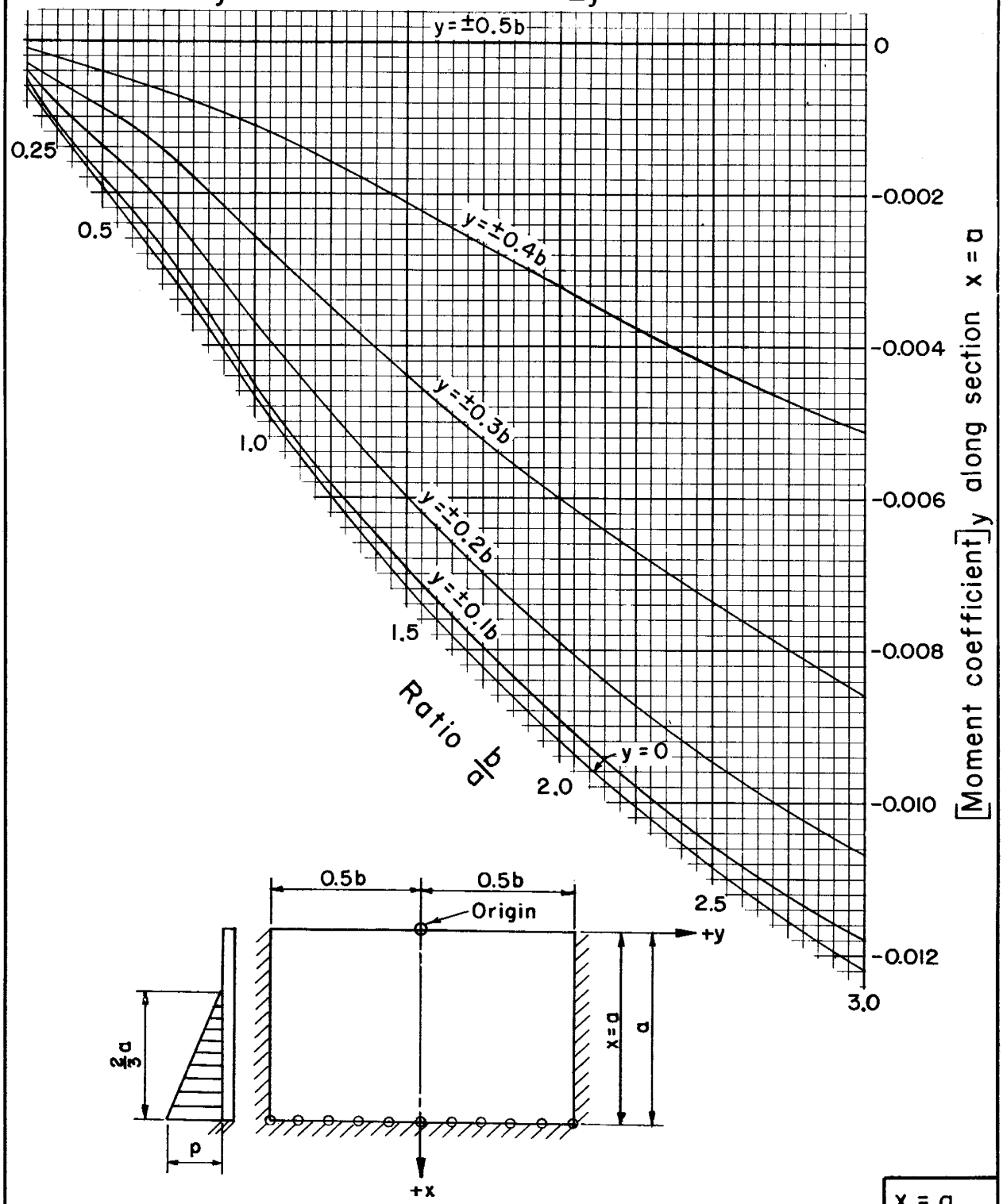
x = 0.8a

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES- 104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 26 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with $^2/_3$ hydrostatic load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = a

Horizontal moment determines tension in horizontal steel

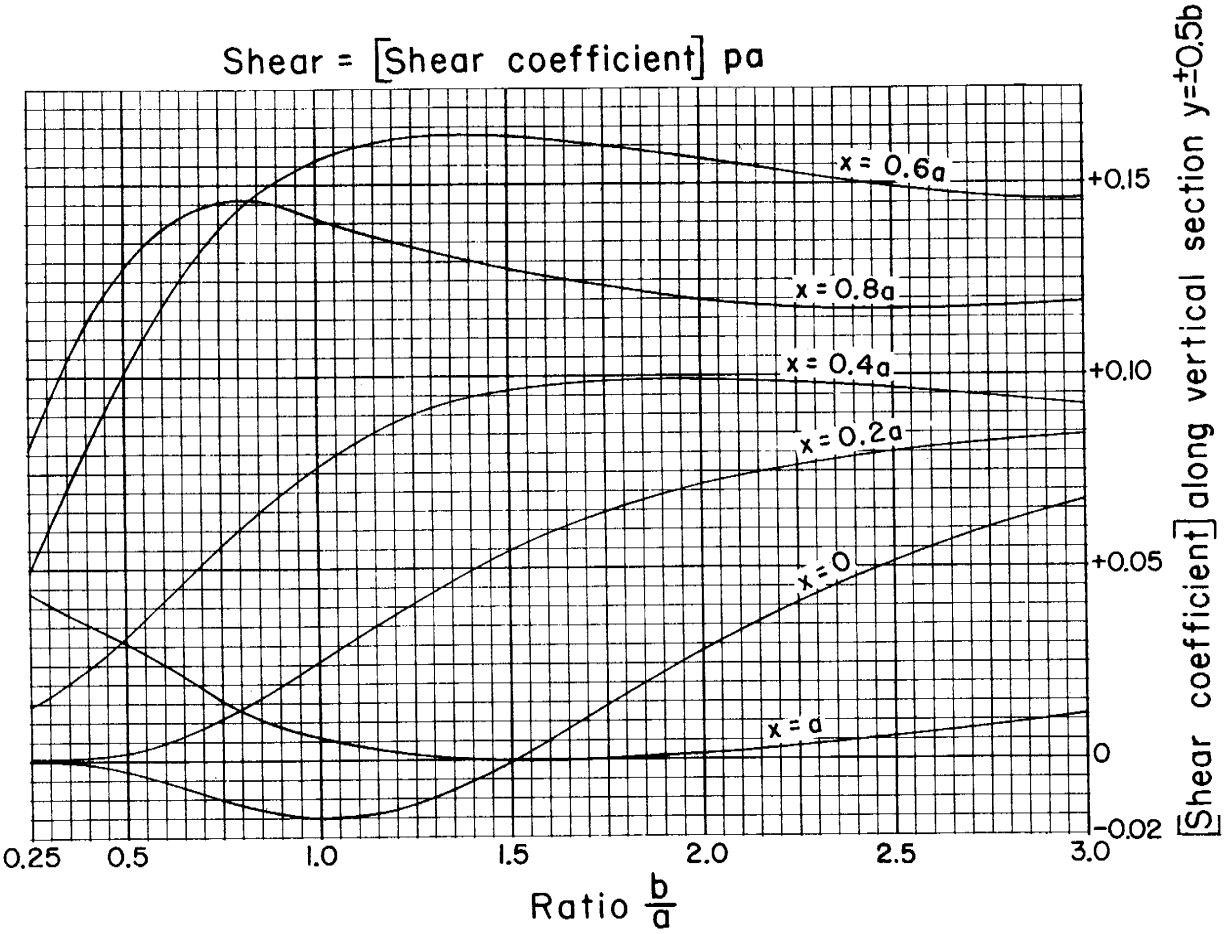
$$M_y = [\text{Moment coefficient}]_y \, pa^2$$

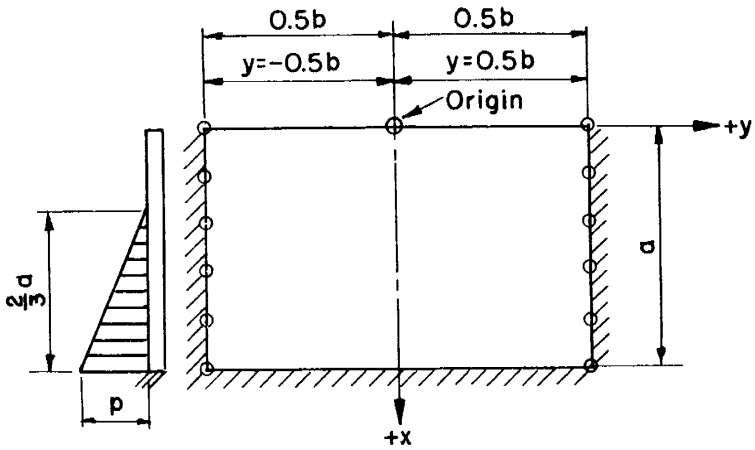


x = a

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES- 104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 27 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN : Rectangular slabs with 2/3 hydrostatic load; coefficients for shear at fifth points on fixed side edges $y = \pm 0.5b$



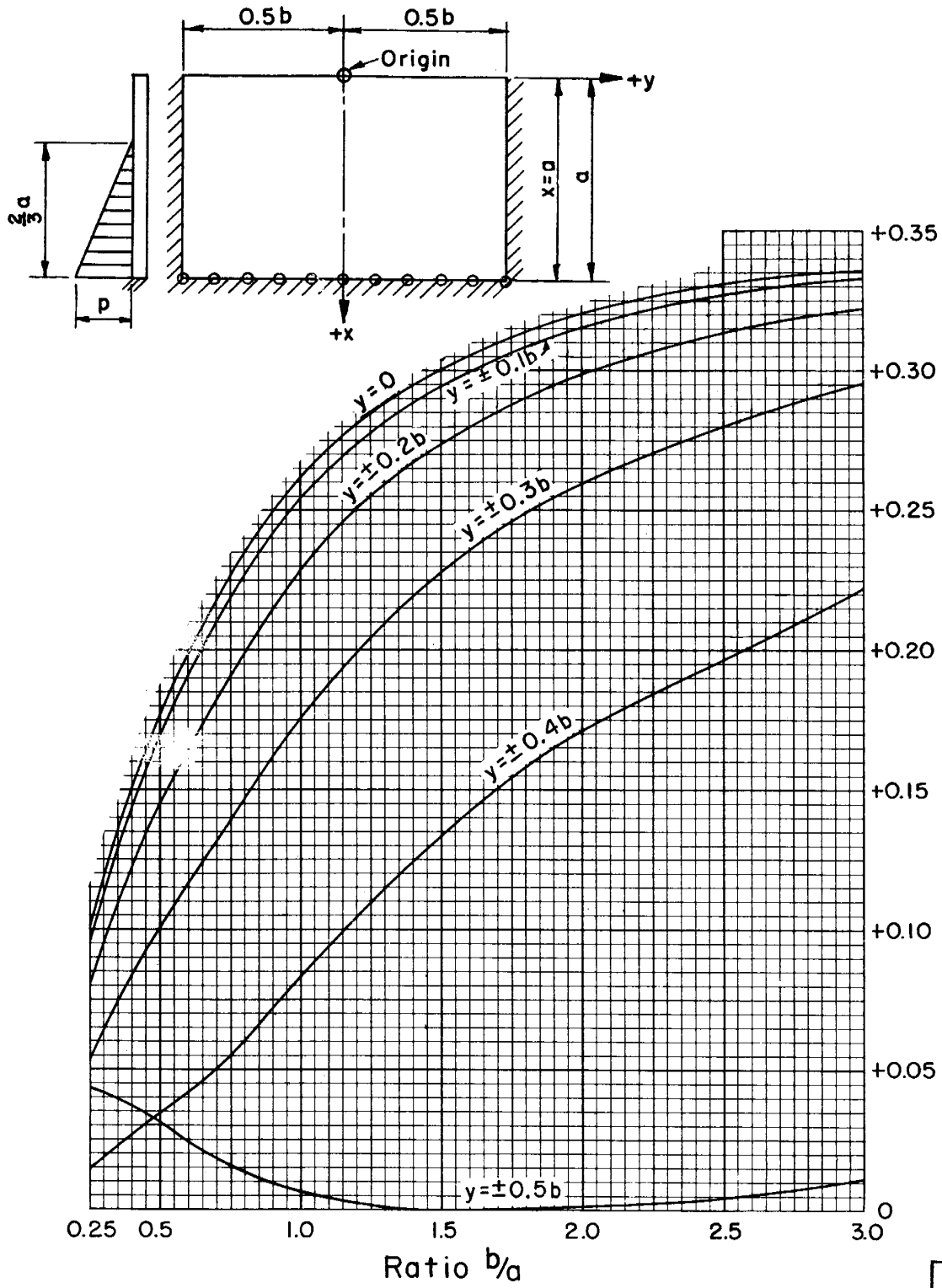


$y = \pm 0.5b$

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 28 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with $^2/_3$ hydrostatic load; coefficients for shear at tenth points on fixed bottom edge x = a

Shear = [Shear coefficient] pa



x = a

REFERENCE

U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES-104
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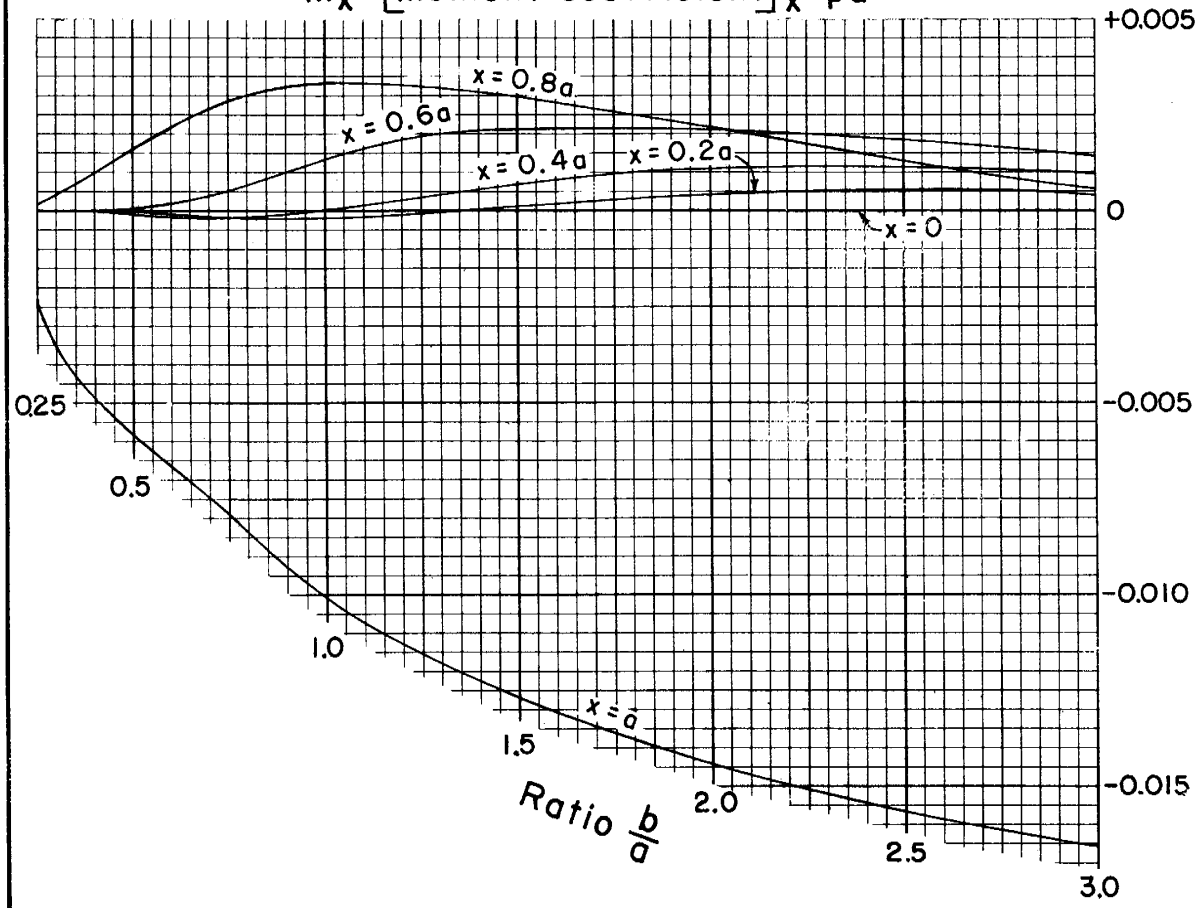
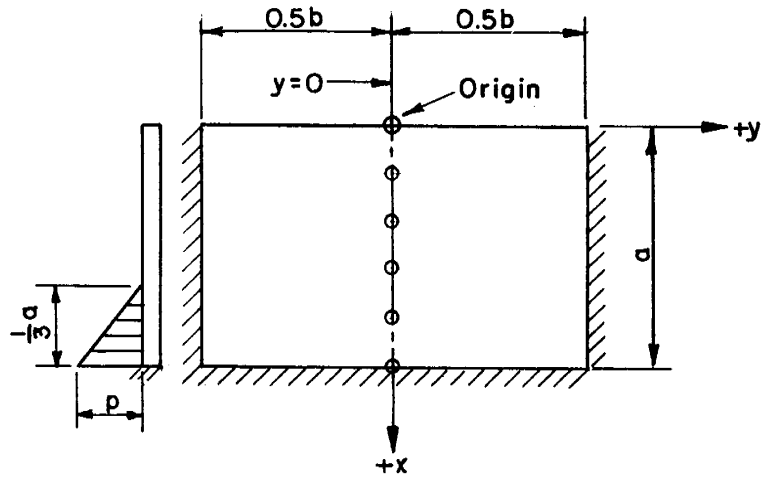
DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with 1/3 hydrostatic load; coefficients for vertical moment, M_x, at fifth points on vertical slice y = 0

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \; pa^2$





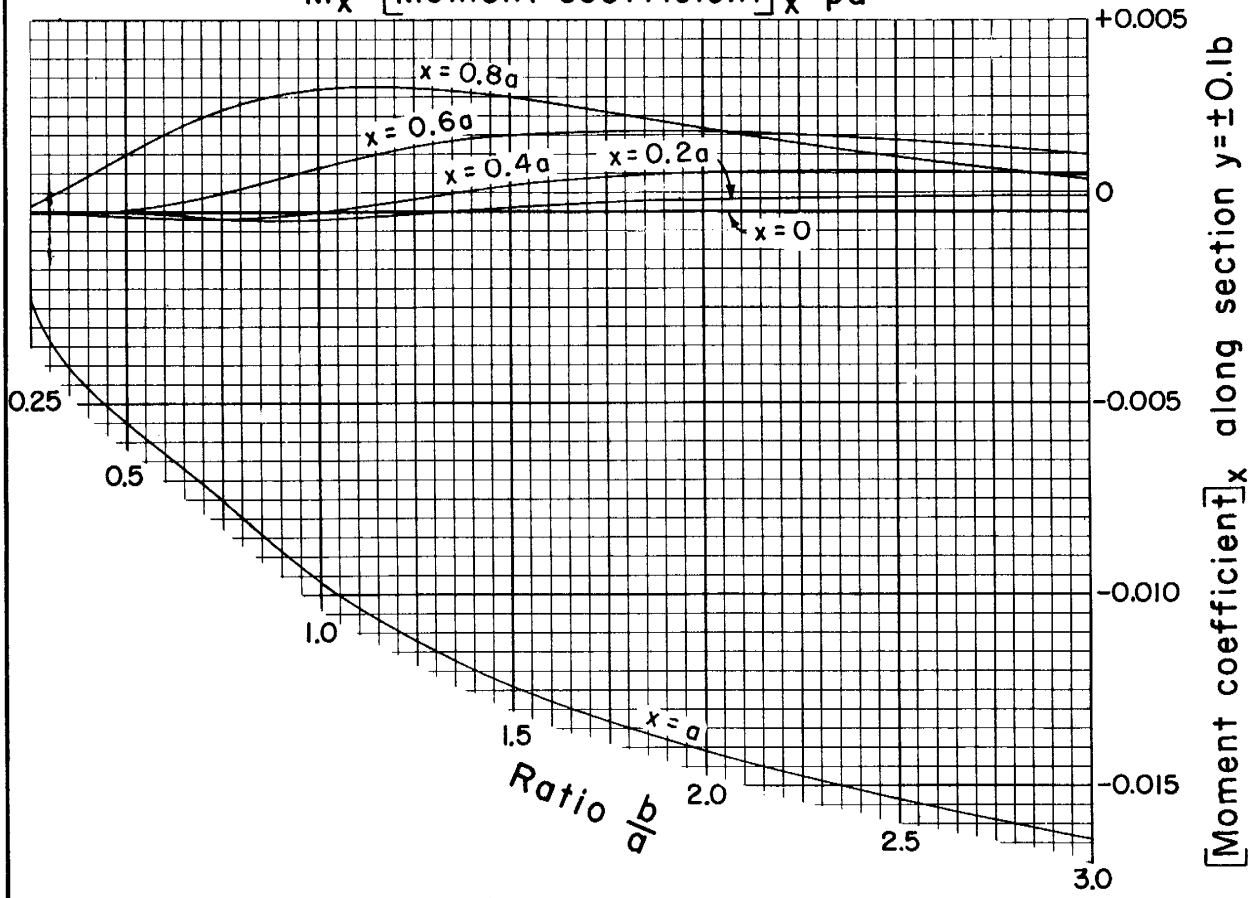
y = 0

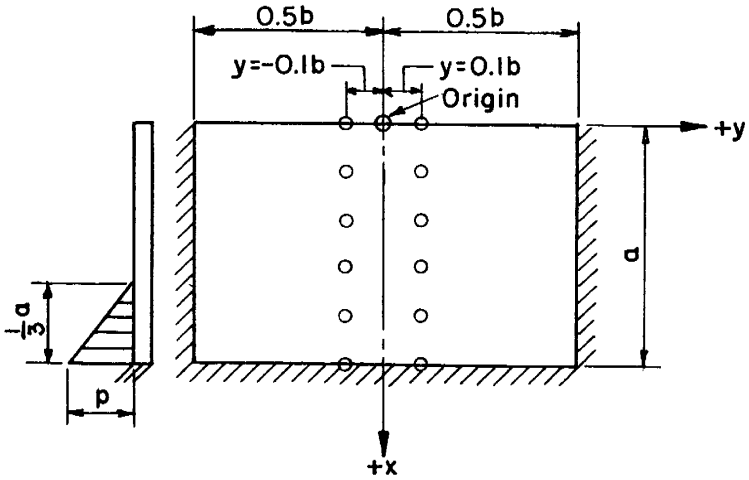
REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 30 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with 1/3 hydrostatic load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.1b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \; pa^2$





$y = \pm 0.1b$

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
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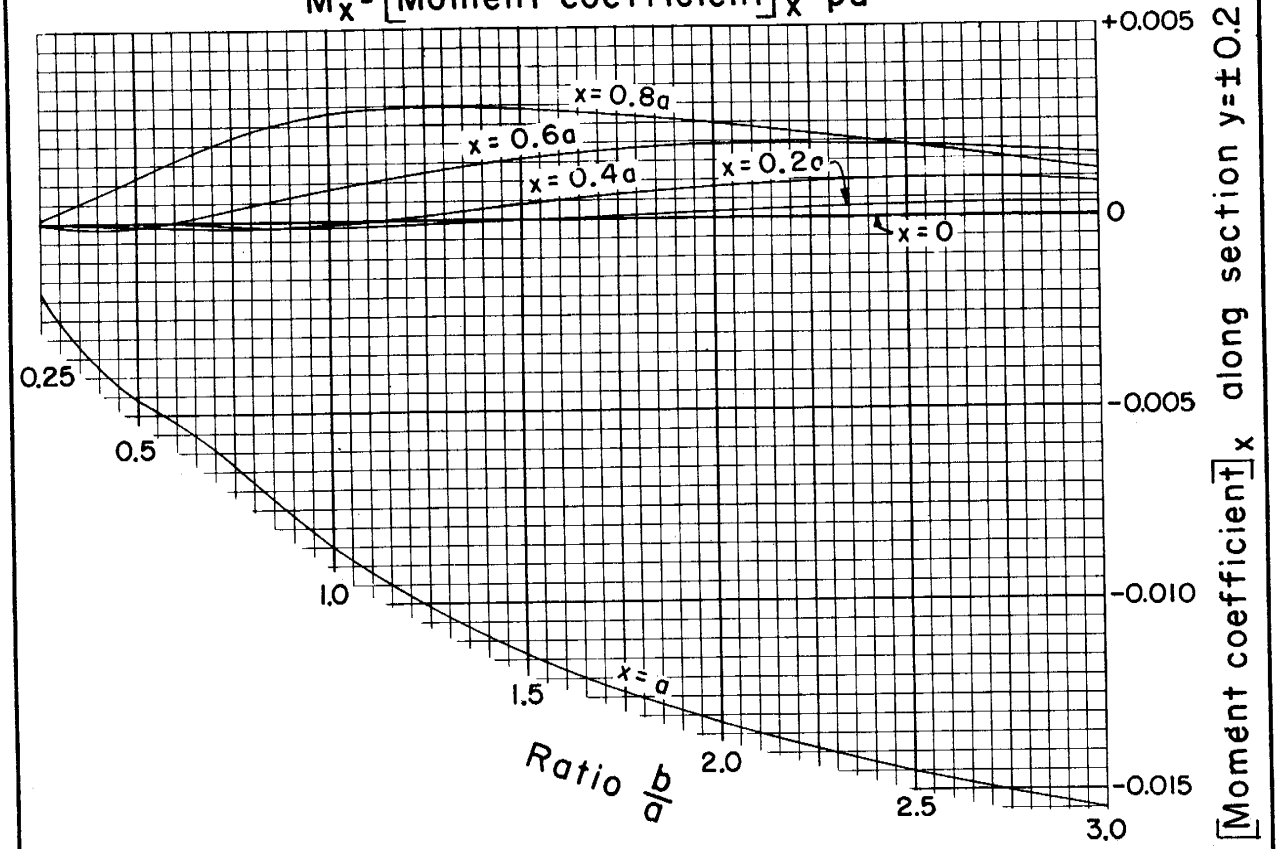
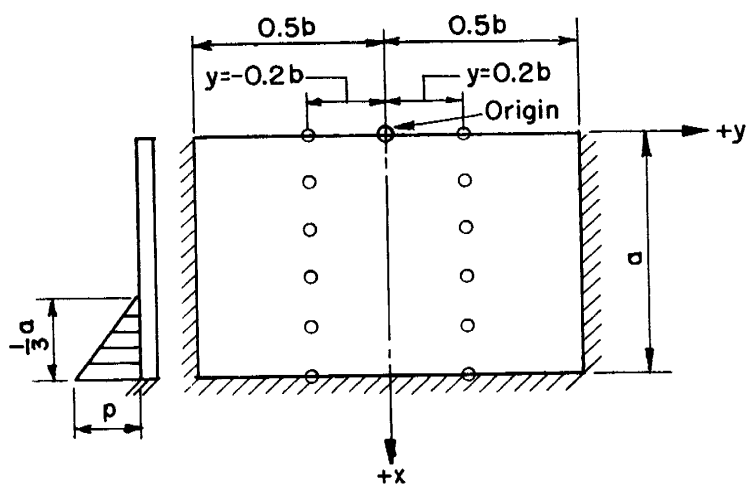
STRUCTURAL DESIGN : Rectangular slabs with 1/3 hydrostatic load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.2b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \; pa^2$





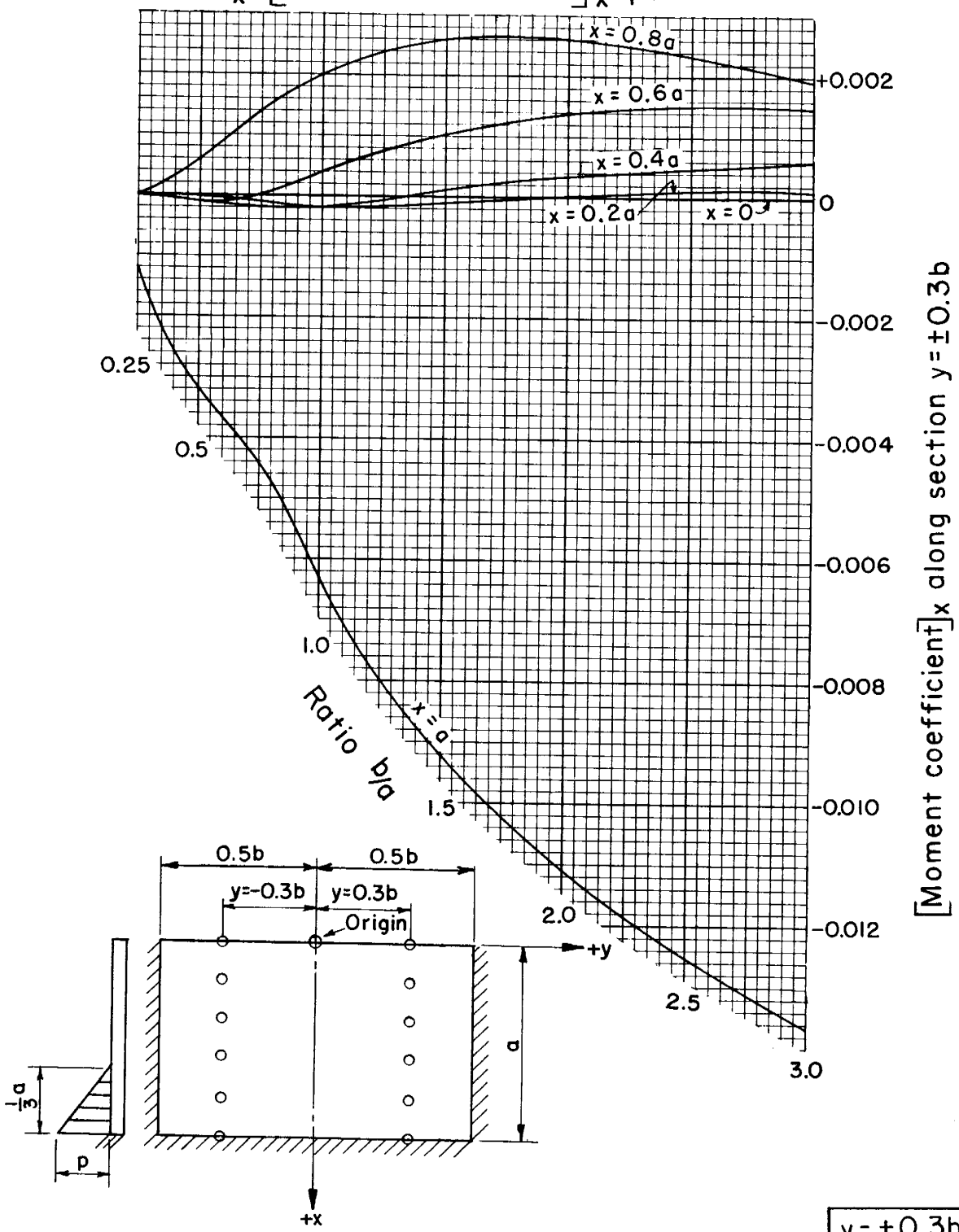
$y = \pm 0.2b$

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 32 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with 1/3 hydrostatic load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.3b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \, pa^2$



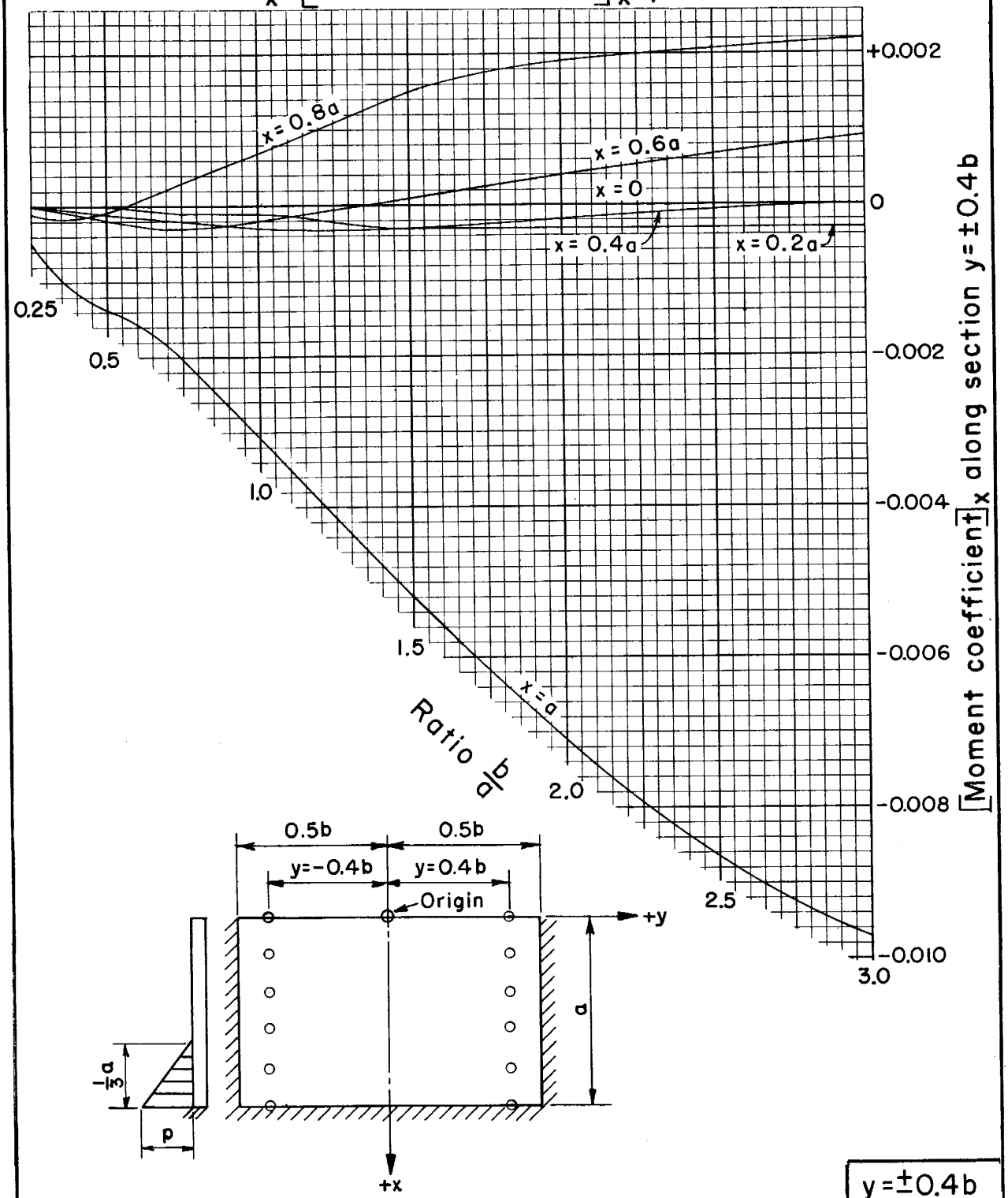
$y = \pm 0.3b$

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 33 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN : Rectangular slabs with 1/3 hydrostatic load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.4b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \; pa^2$



$y = \pm 0.4b$

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 34 OF 85
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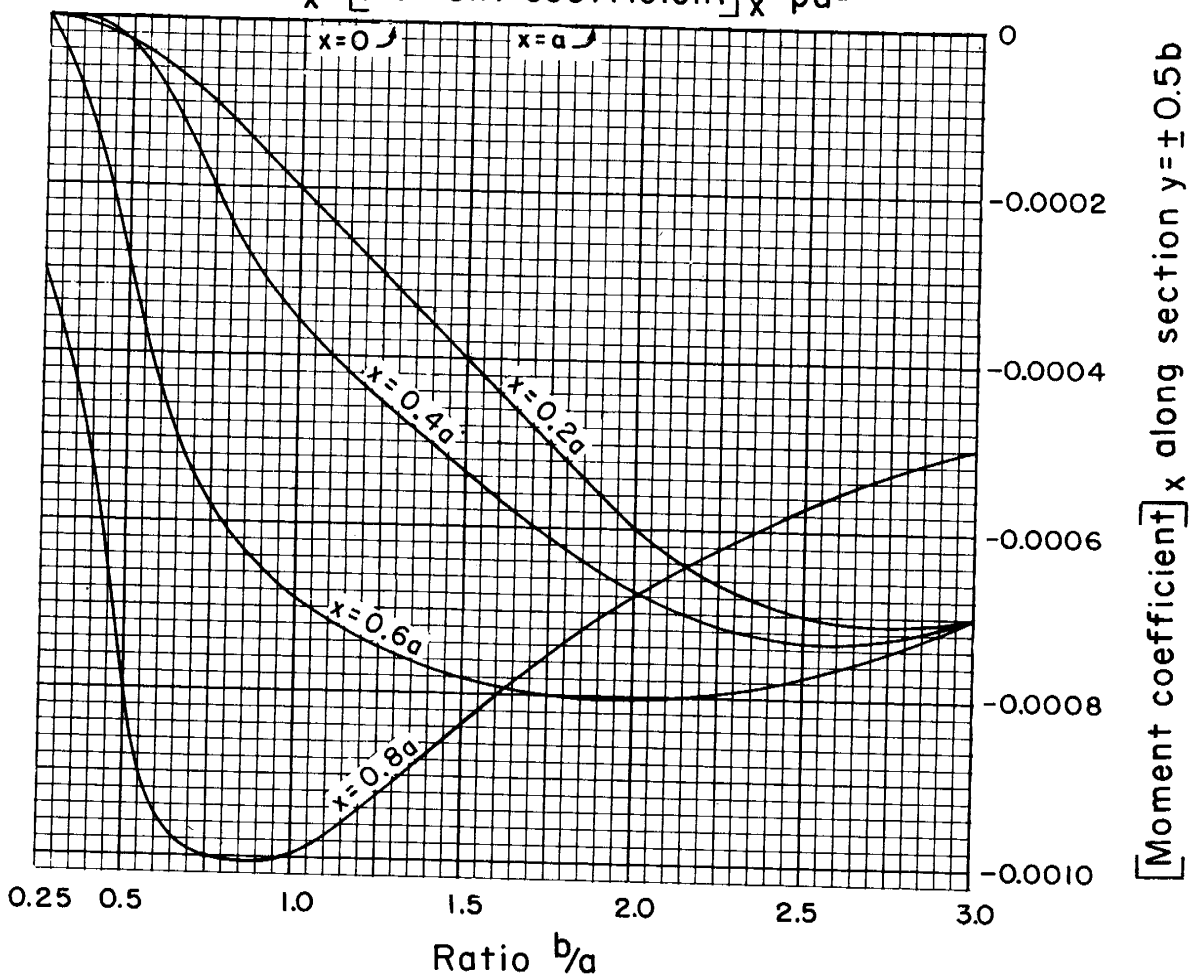
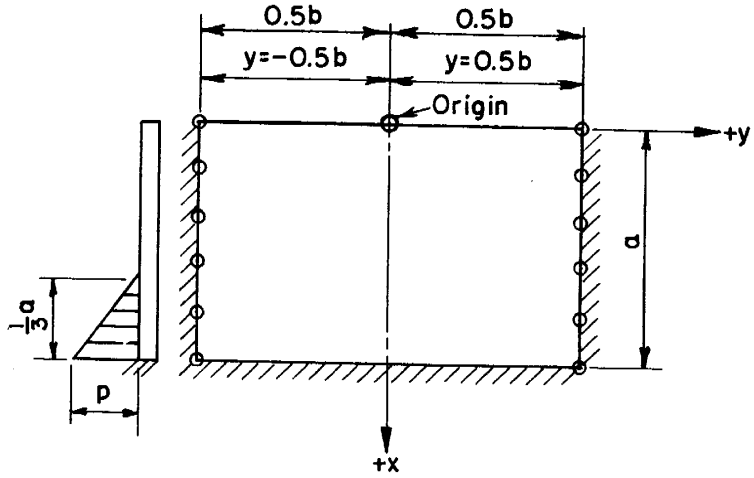
STRUCTURAL DESIGN : Rectangular slabs with 1/3 hydrostatic load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.5b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \; pa^2$





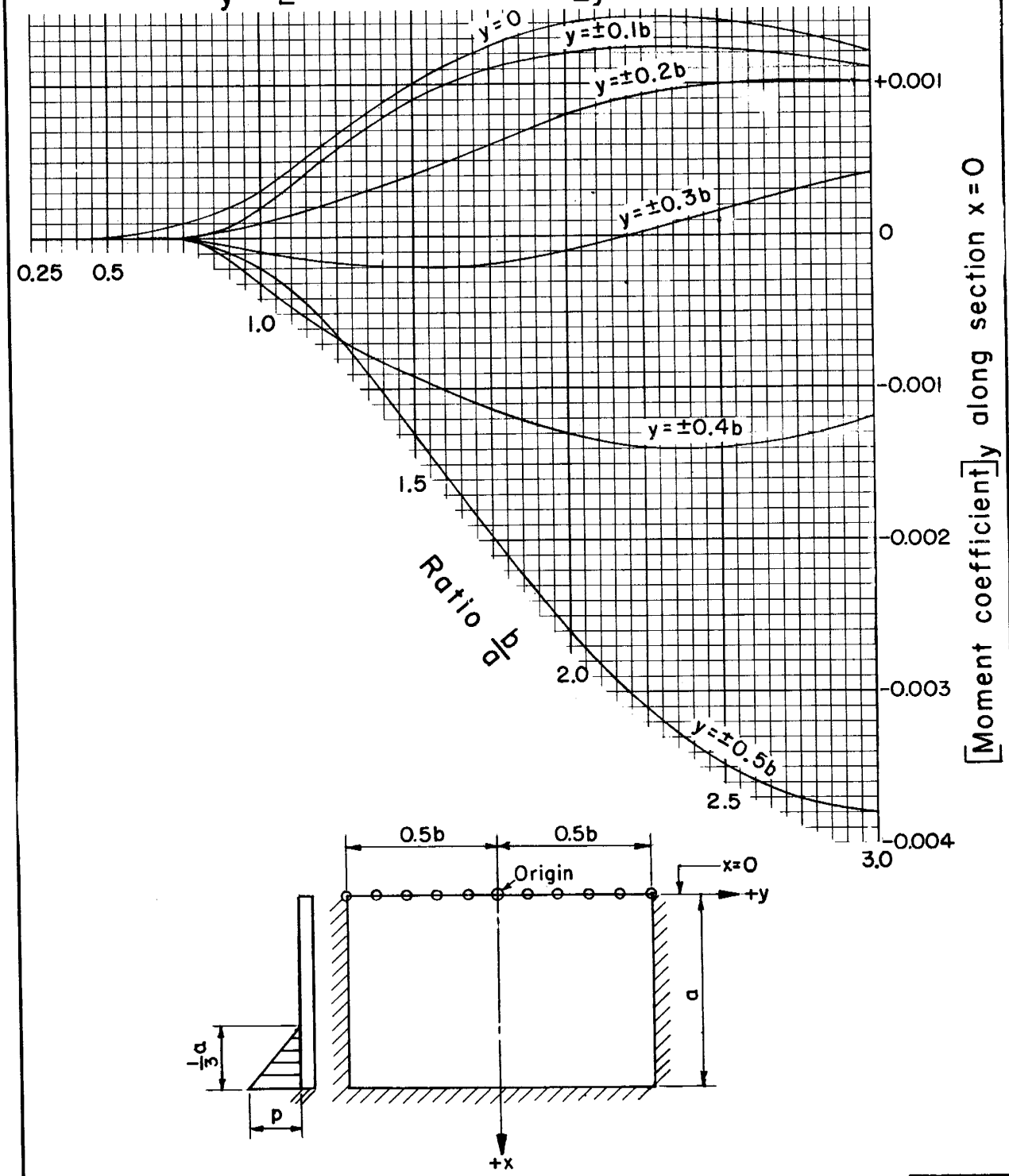
$y = \pm 0.5b$

REFERENCE		
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - DESIGN SECTION	STANDARD DWG. NO. ES-104 SHEET 35 OF 85 DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with 1/3 hydrostatic load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = 0

Horizontal moment determines tension in horizontal steel

M_y = [Moment coefficient]$_y$ pa^2



x = 0

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 36 OF 85
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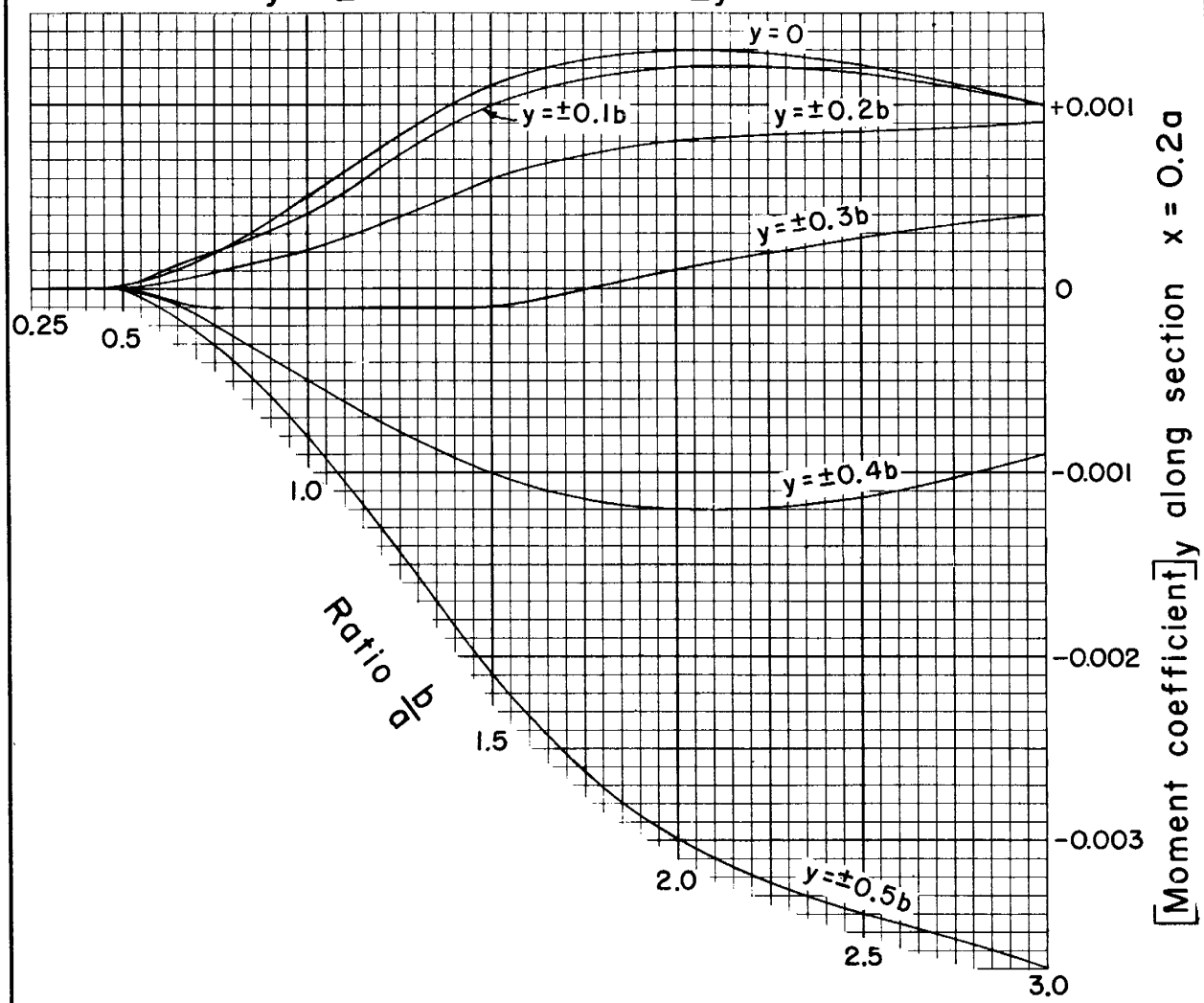
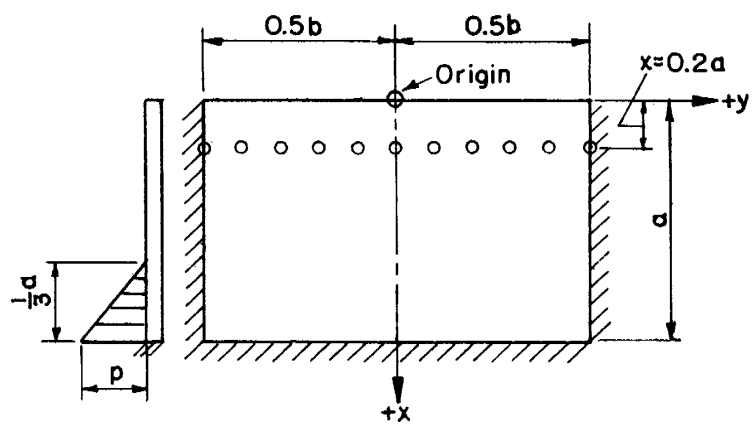
STRUCTURAL DESIGN: Rectangular slabs with 1/3 hydrostatic load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = 0.2a

Horizontal moment determines tension in horizontal steel

M_y = [Moment coefficient]$_y$ pa^2





x = 0.2a

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
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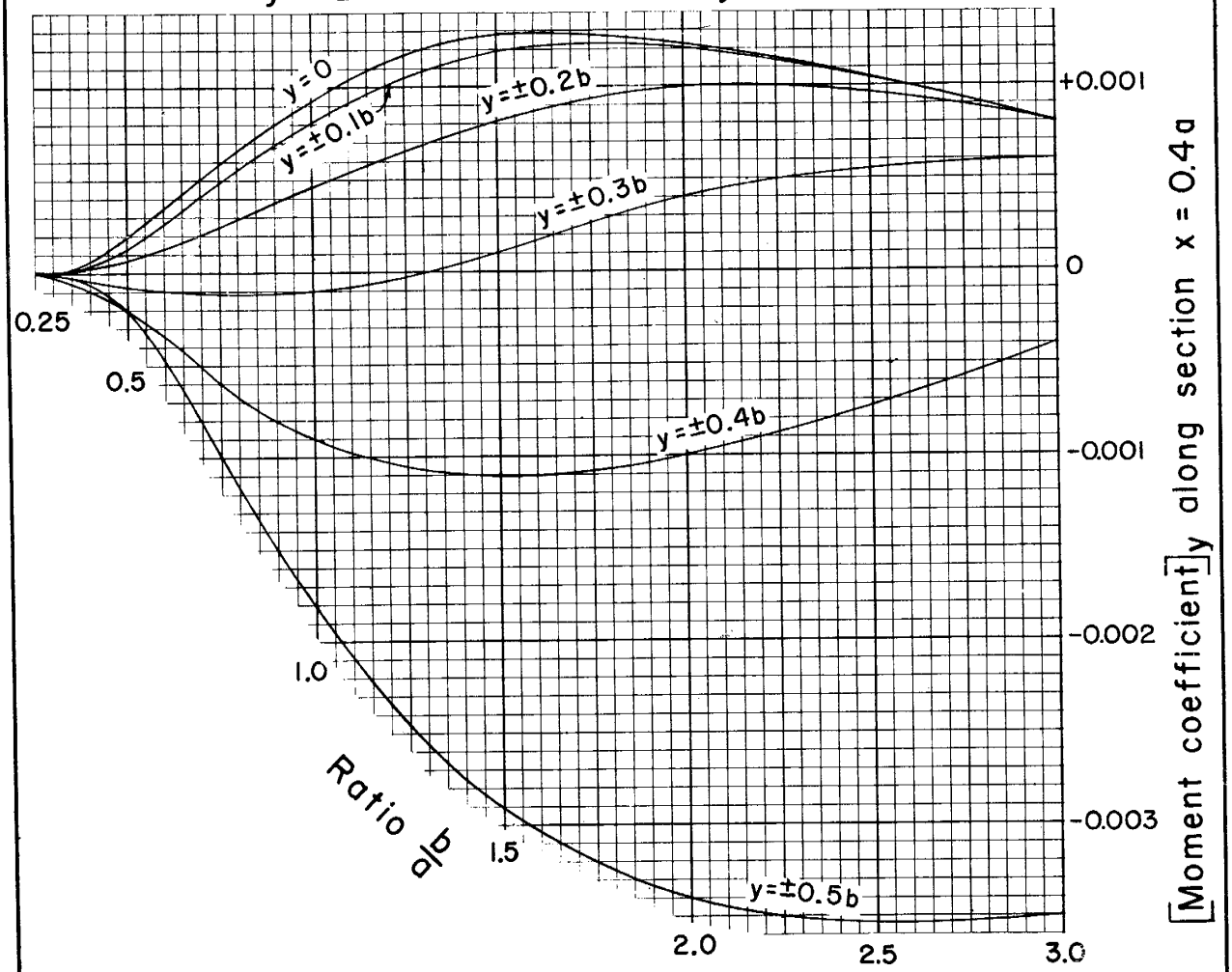
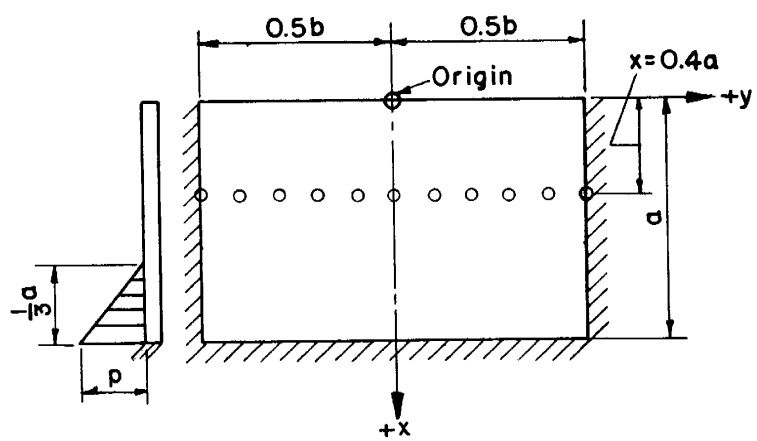
STRUCTURAL DESIGN: Rectangular slabs with ⅓ hydrostatic load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = 0.4a

Horizontal moment determines tension in horizontal steel

$$M_y = [\text{Moment coefficient}]_y \, pa^2$$





x = 0.4a

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
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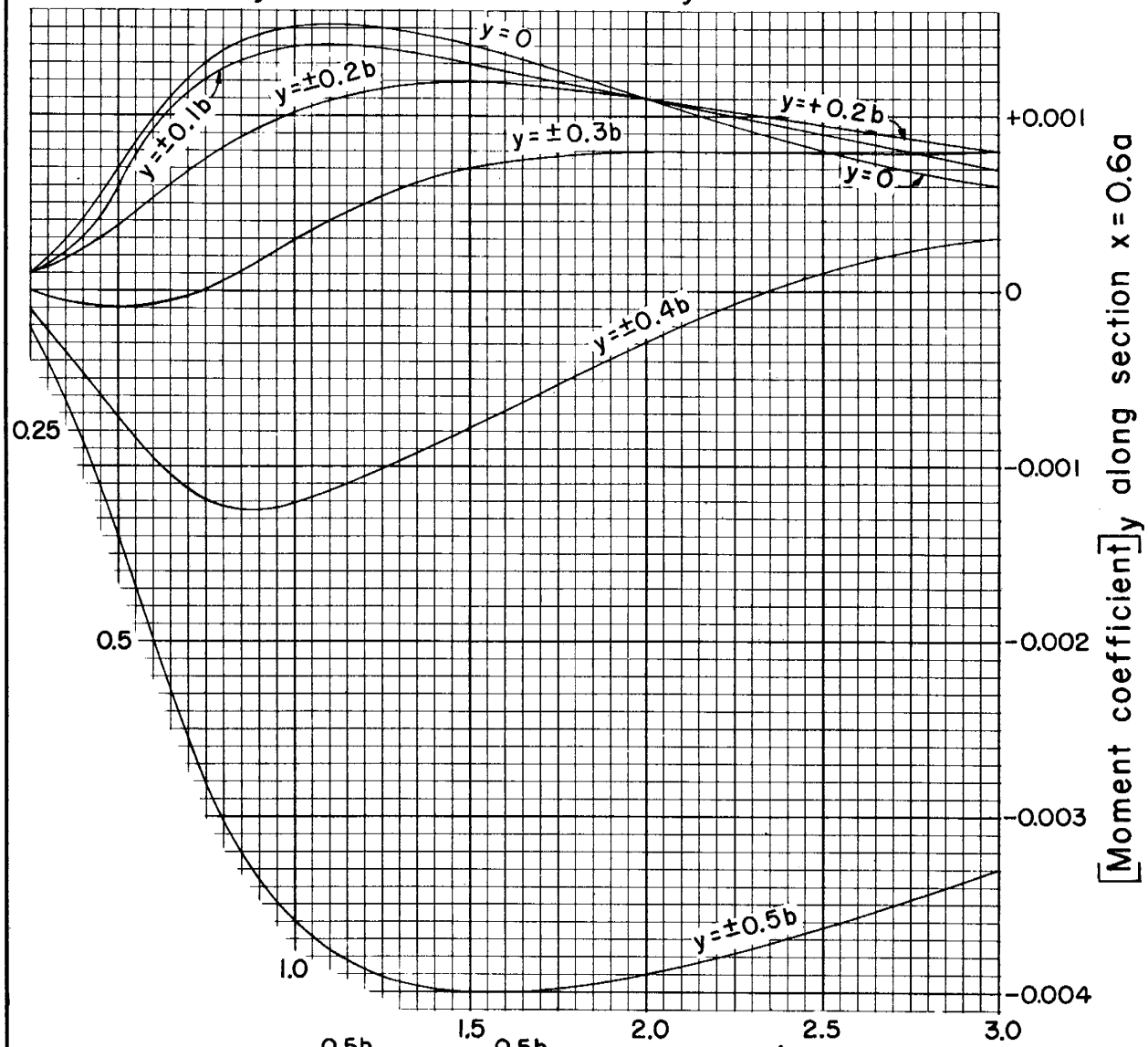
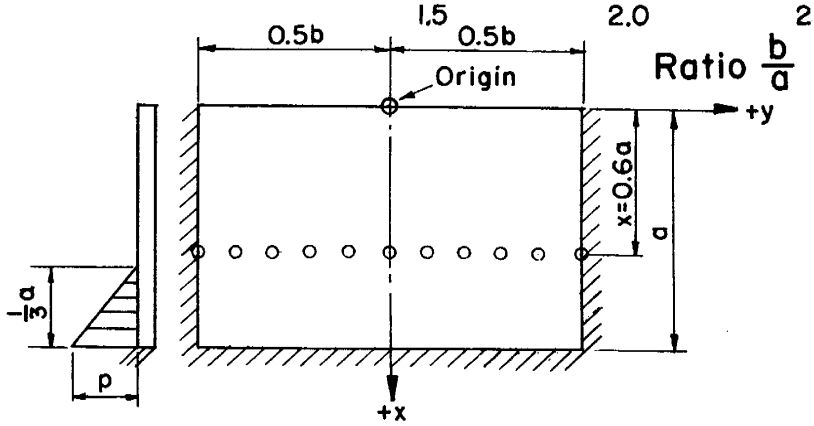
STRUCTURAL DESIGN: Rectangular slabs with 1/3 hydrostatic load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = 0.6a

Horizontal moment determines tension in horizontal steel

M_y = [Moment coefficient]$_y$ pa^2





x = 0.6a

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
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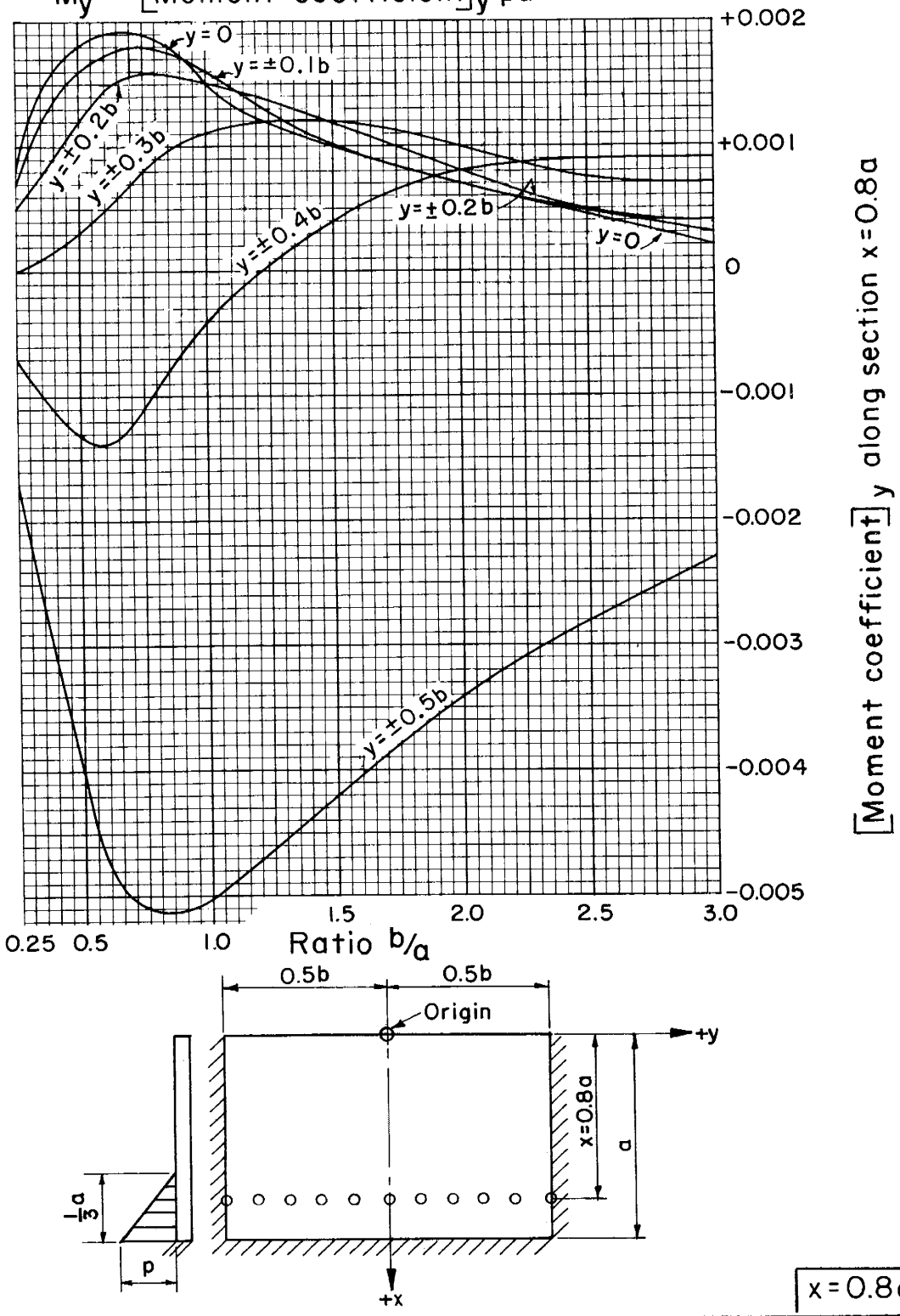
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with 1/3 hydrostatic load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = 0.8a

Horizontal moment determines tension in horizontal steel

M_y = [Moment coefficient]$_y$ pa^2



x = 0.8a

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
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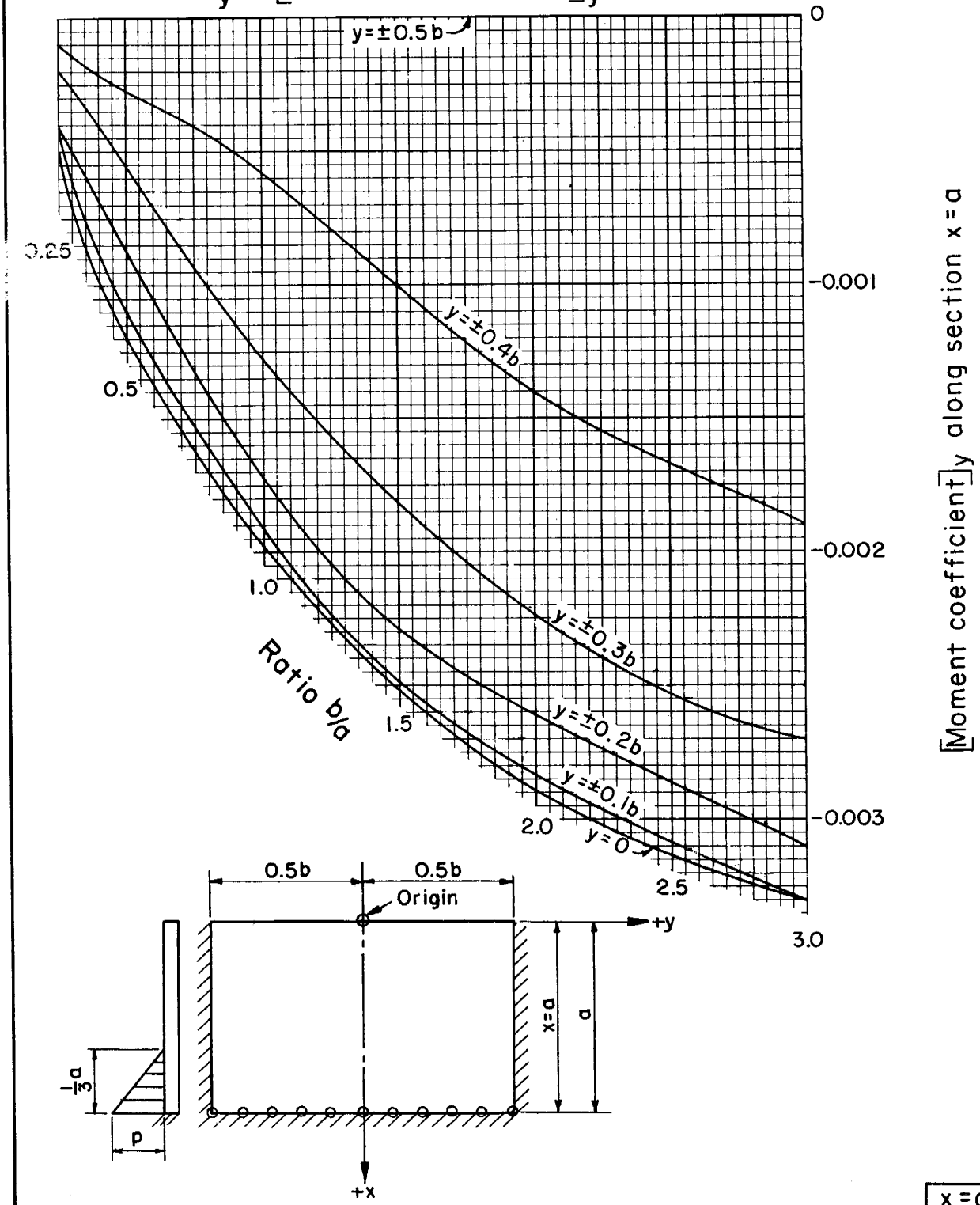
STRUCTURAL DESIGN: Rectangular slabs with ⅓ hydrostatic load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = a

Horizontal moment determines tension in horizontal steel

$$M_y = [\text{Moment coefficient}]_y \, pa^2$$



x = a

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
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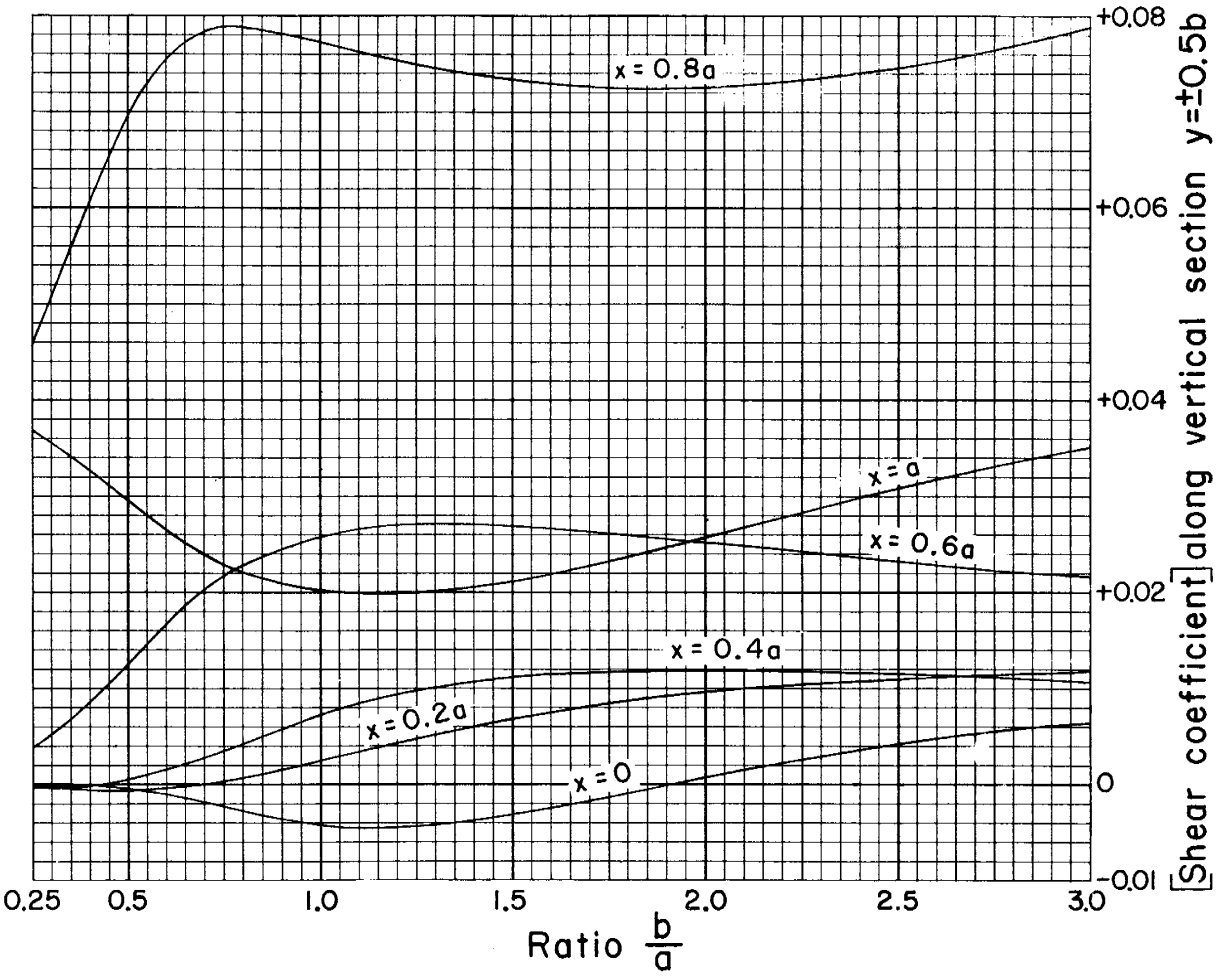
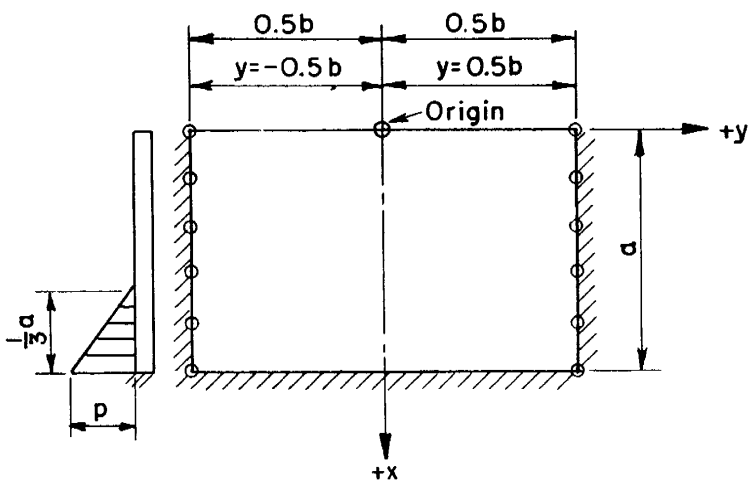
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with 1/3 hydrostatic load; coefficients for shear at fifth points on fixed side edges y = ±0.5b

Shear = [Shear coefficient] pa





y = ±0.5b

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
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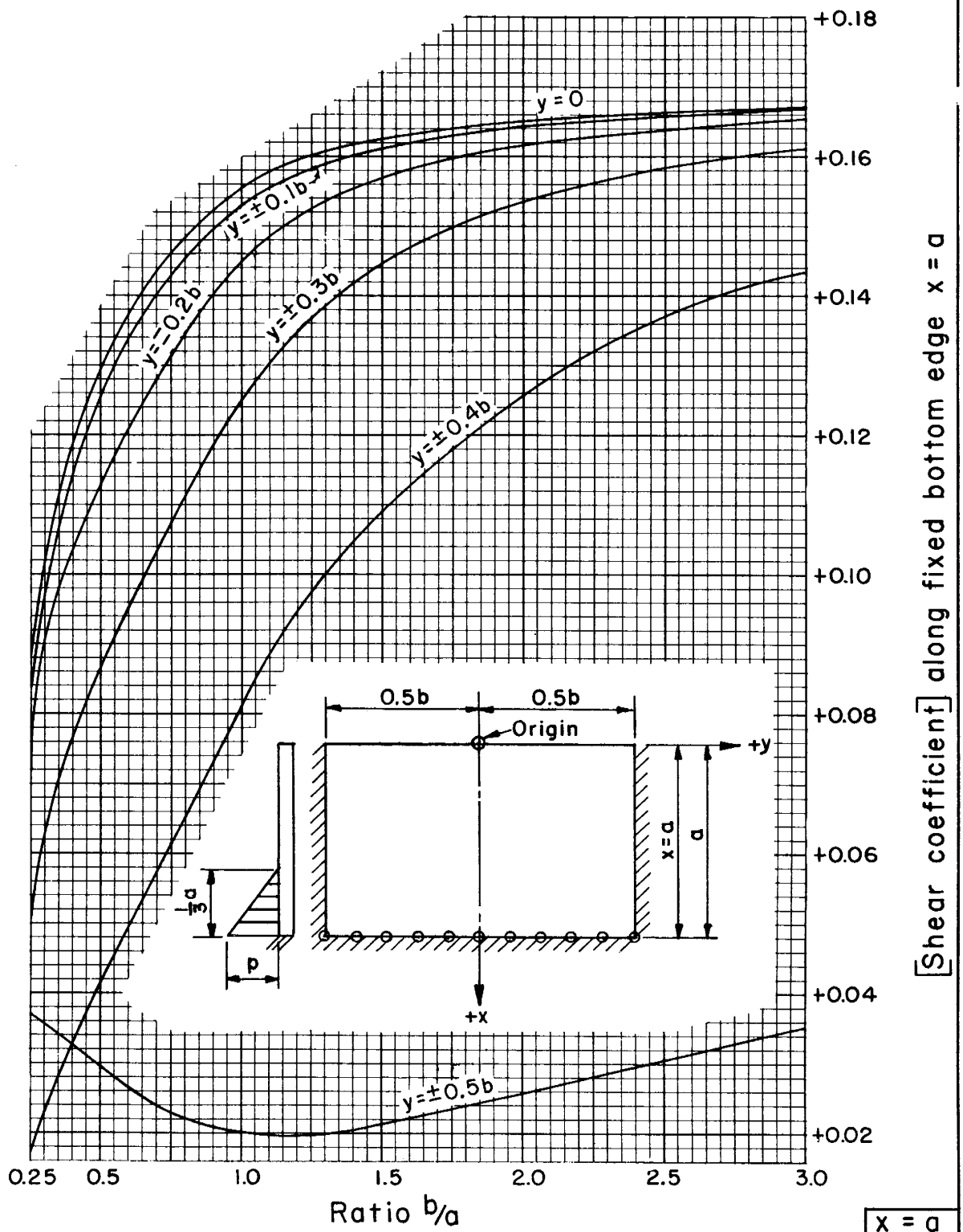
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with ⅓ hydrostatic load; coefficients for shear at tenth points on fixed bottom edge x = a

Shear = [Shear coefficient] pa



x = a

REFERENCE

U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

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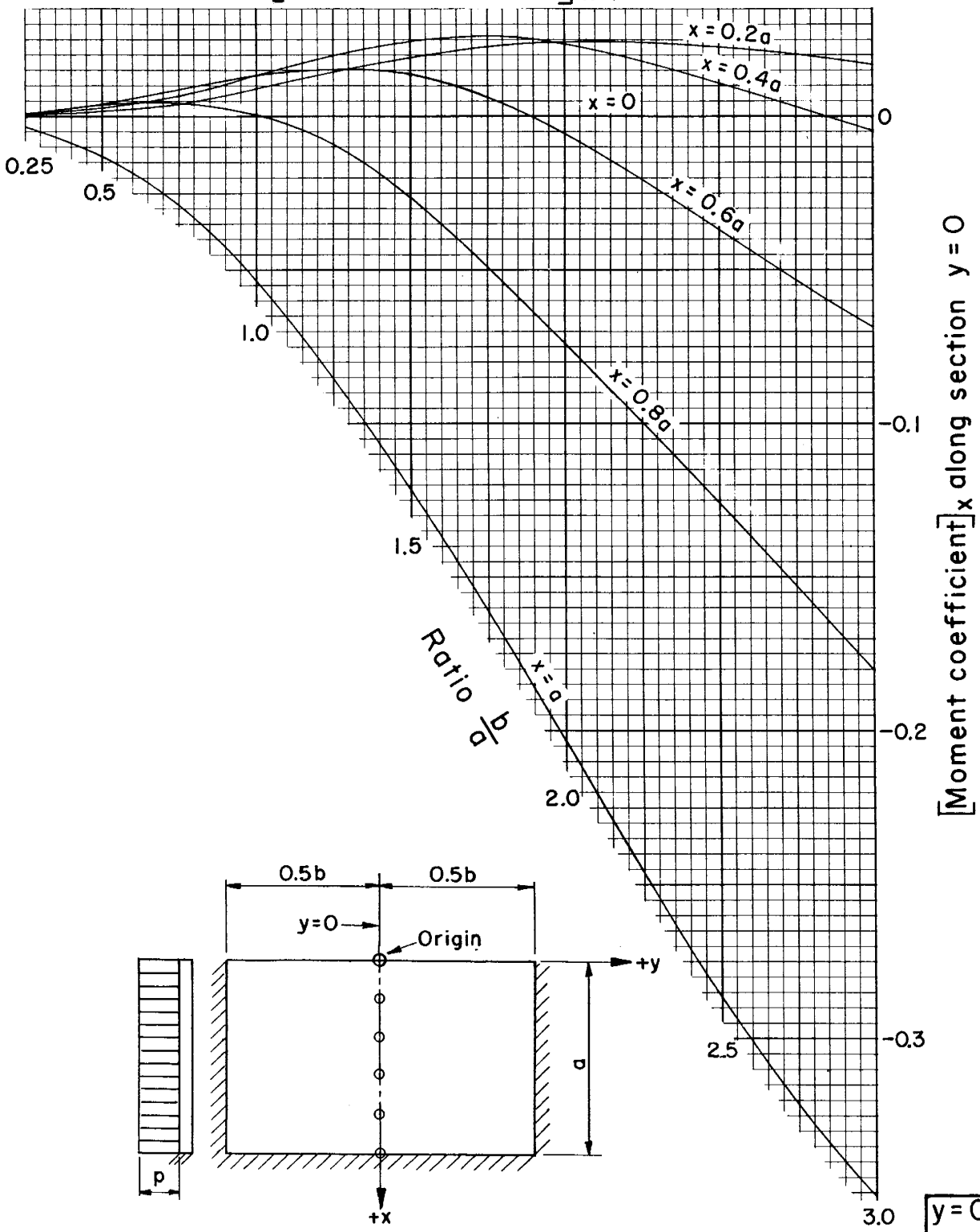
STRUCTURAL DESIGN : Rectangular slabs with uniform load; coefficients for vertical moment, M_x, at fifth points on vertical slice y = 0

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \, pa^2$



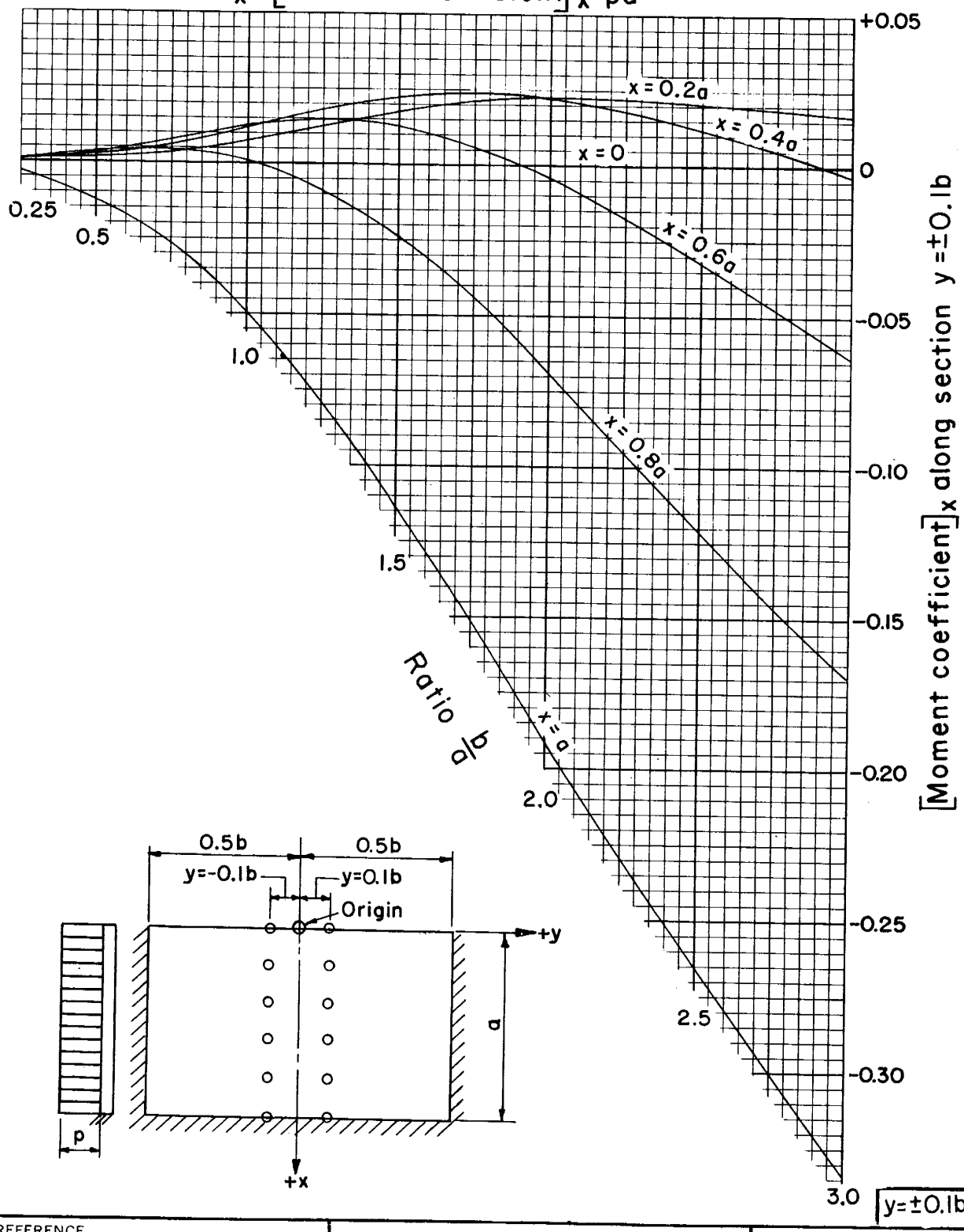
y = 0

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 44 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN : Rectangular slabs with uniform load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.1b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \, pa^2$



REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
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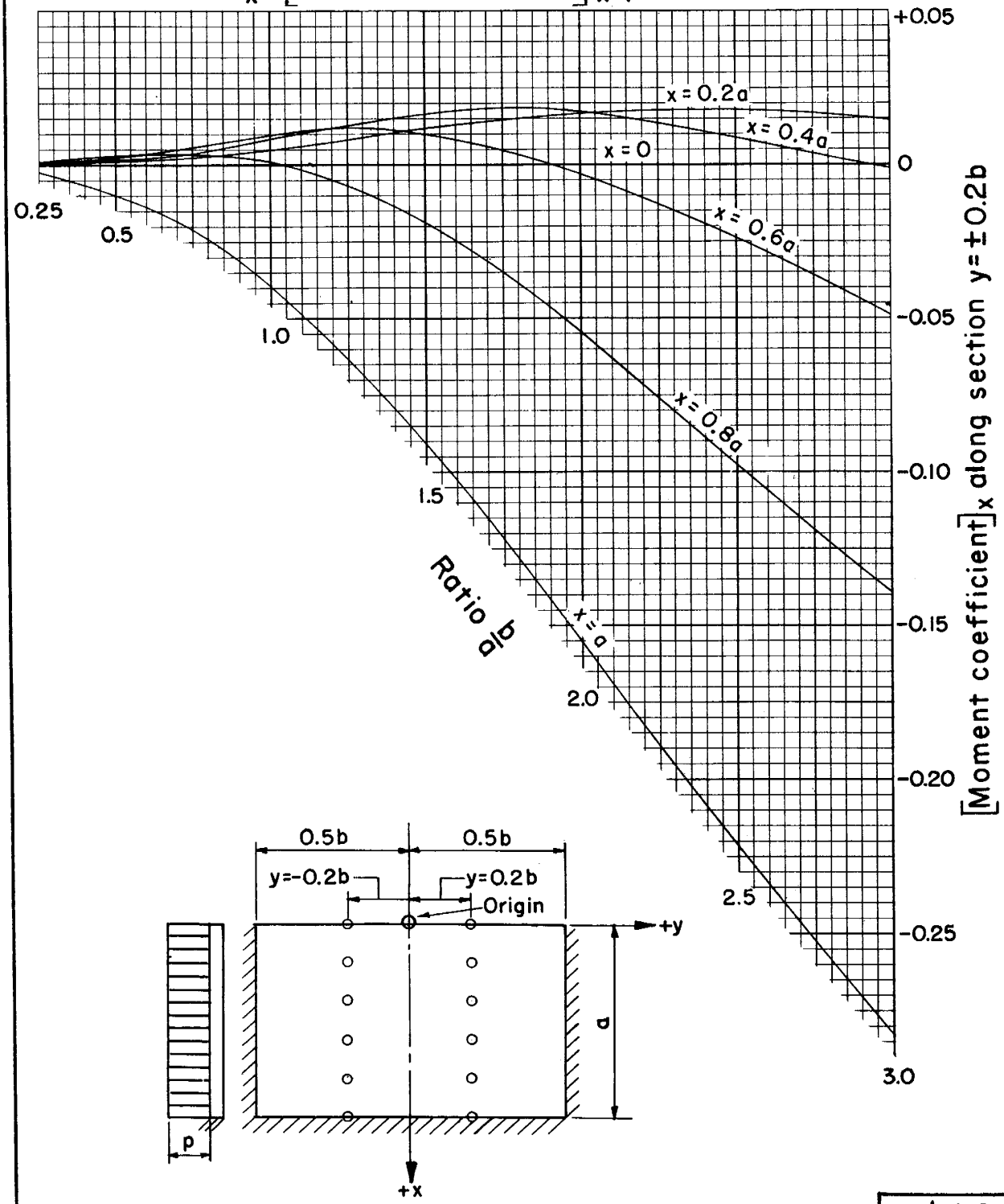
STRUCTURAL DESIGN : Rectangular slabs with uniform load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.2b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \, pa^2$



$y = \pm 0.2b$

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
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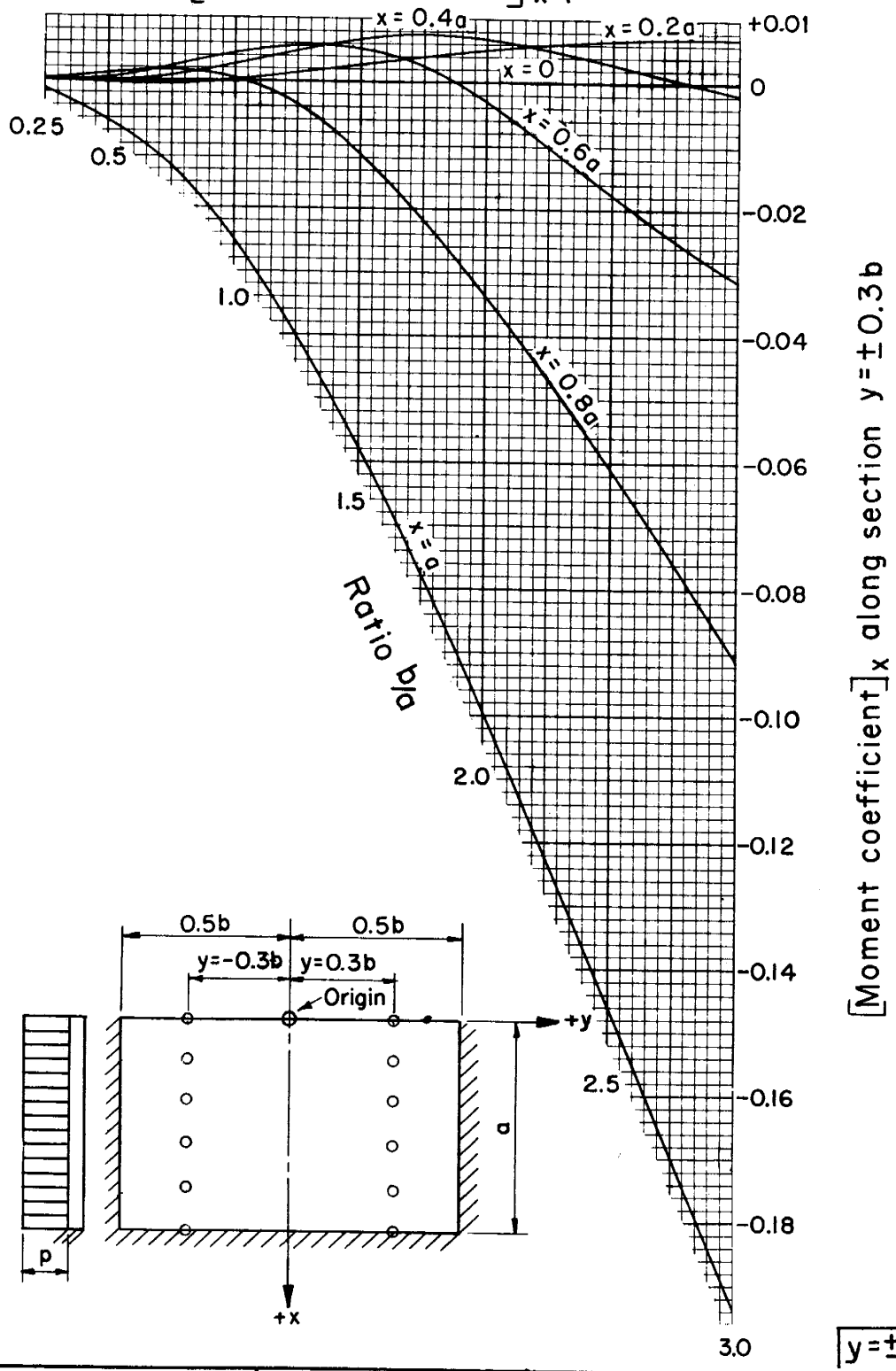
STRUCTURAL DESIGN : Rectangular slabs with uniform load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.3b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \, pa^2$



$y = \pm 0.3b$

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
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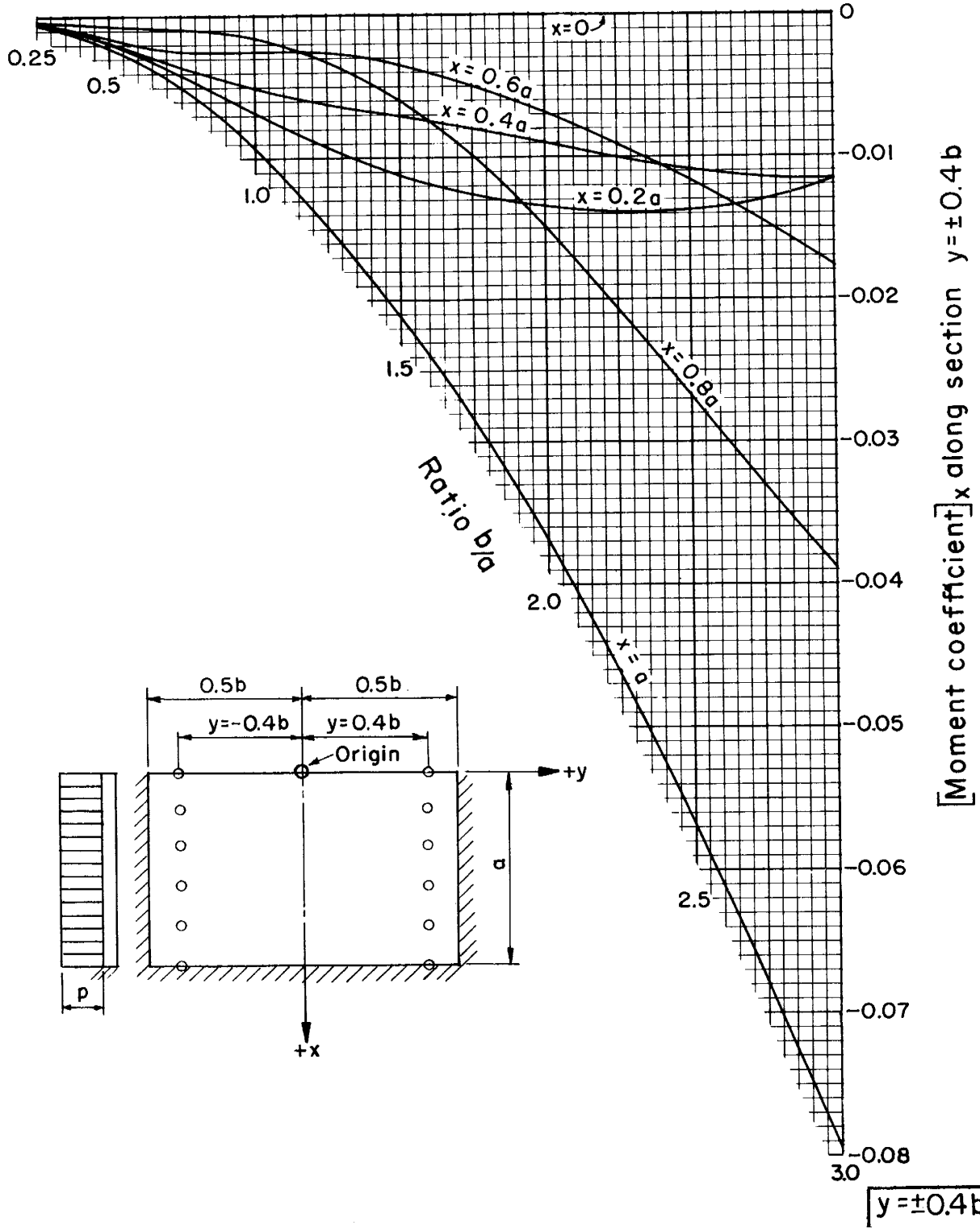
STRUCTURAL DESIGN : Rectangular slabs with uniform load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.4b$

Vertical moment determines tension in vertical steel

$$M_x = [\text{Moment coefficient}]_x \, pa^2$$



$y = \pm 0.4b$

REFERENCE

U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954

U. S. DEPARTMENT OF AGRICULTURE
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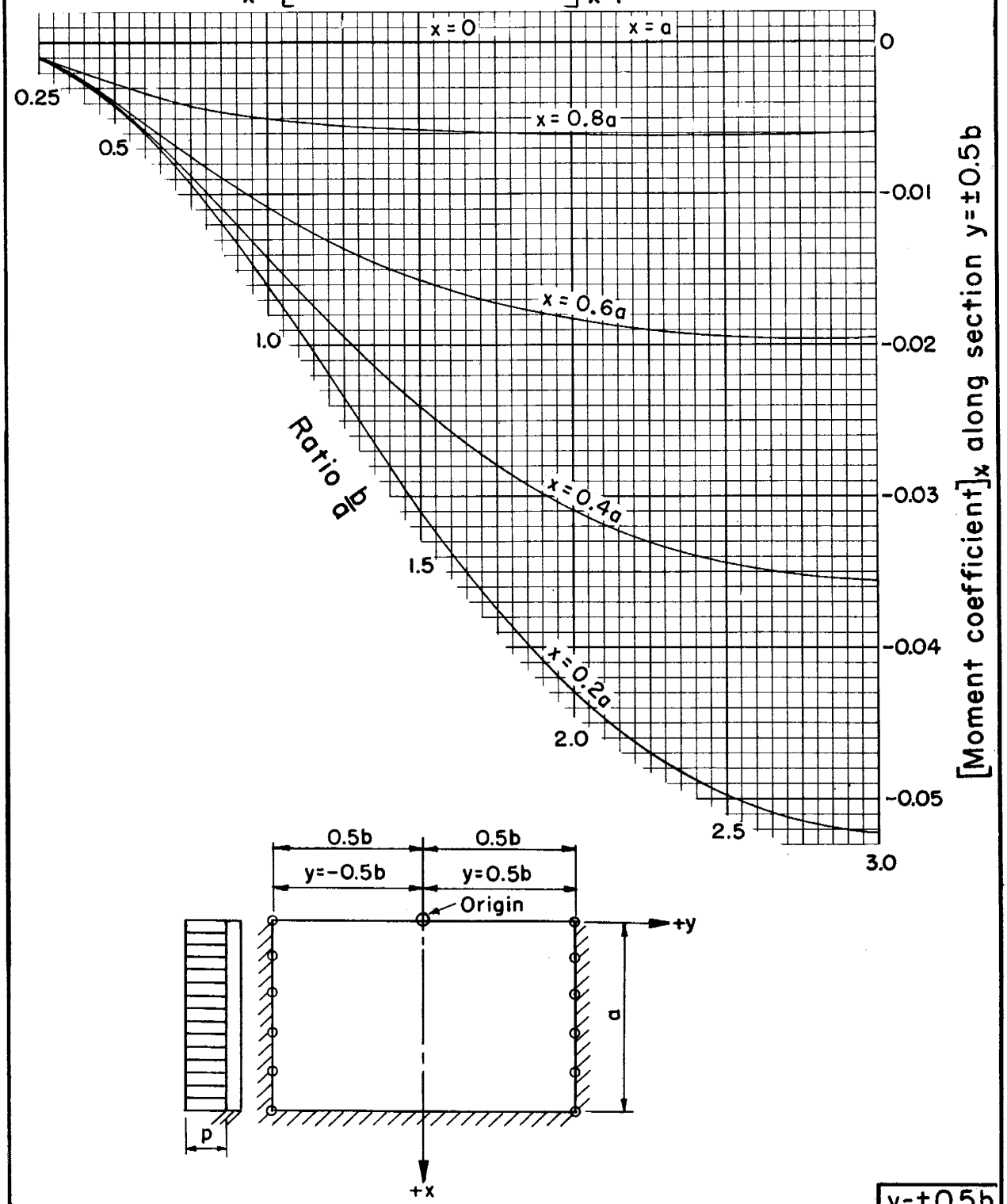
STRUCTURAL DESIGN : Rectangular slabs with uniform load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.5b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \times pa^2$



$y = \pm 0.5b$

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
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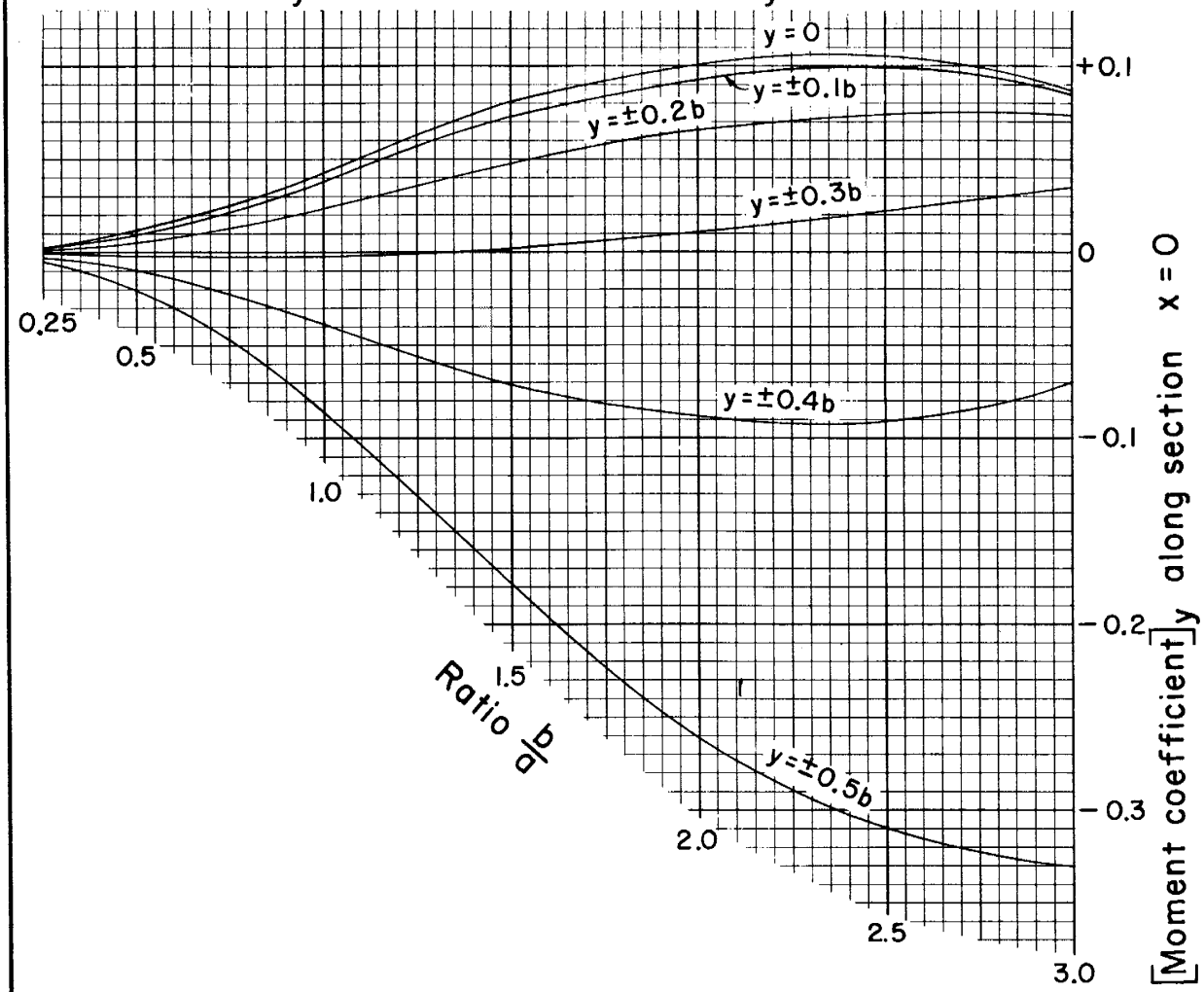
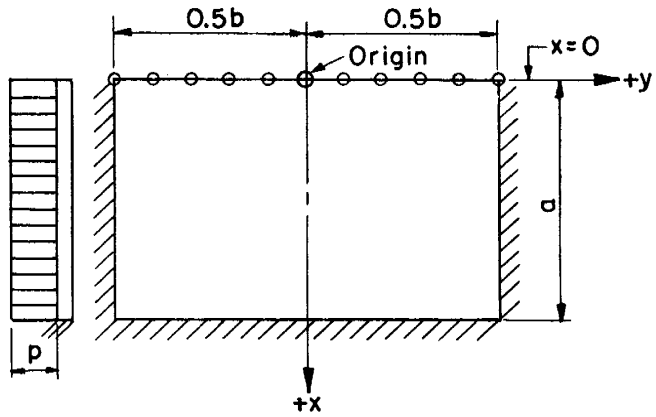
STRUCTURAL DESIGN: Rectangular slabs with uniform load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = 0

Horizontal moment determines tension in horizontal steel

$M_y = [\text{Moment coefficient}]_y \, pa^2$





x = 0

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
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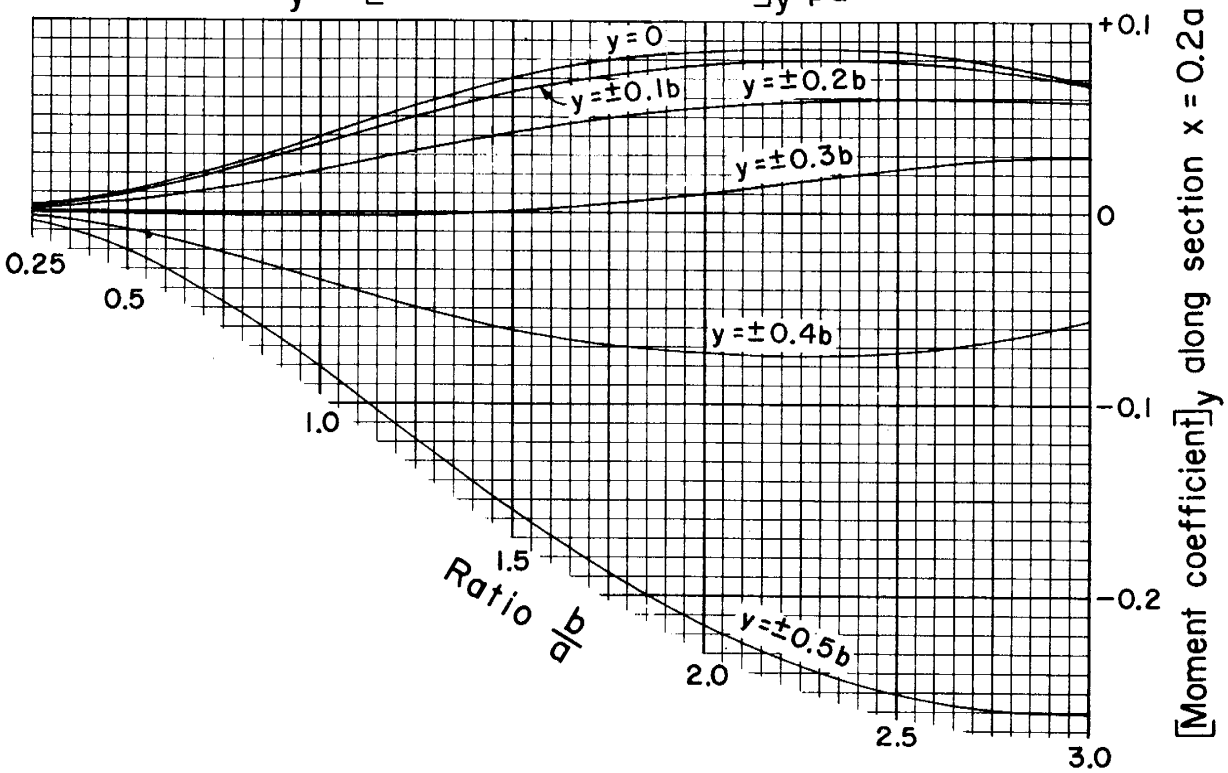
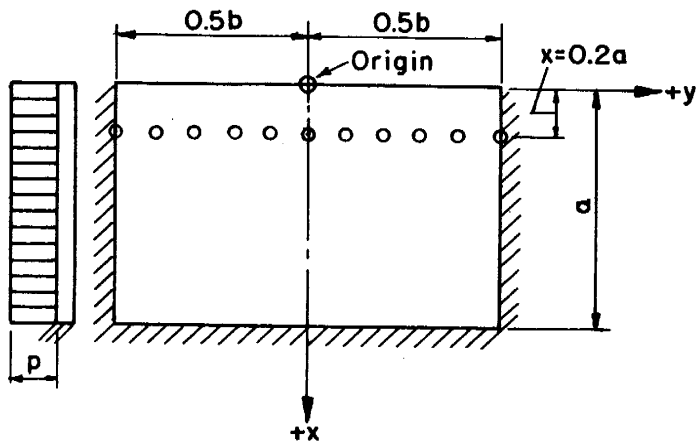
STRUCTURAL DESIGN: Rectangular slabs with uniform load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = 0.2a

Horizontal moment determines tension in horizontal steel

M_y = [Moment coefficient]$_y$ pa^2





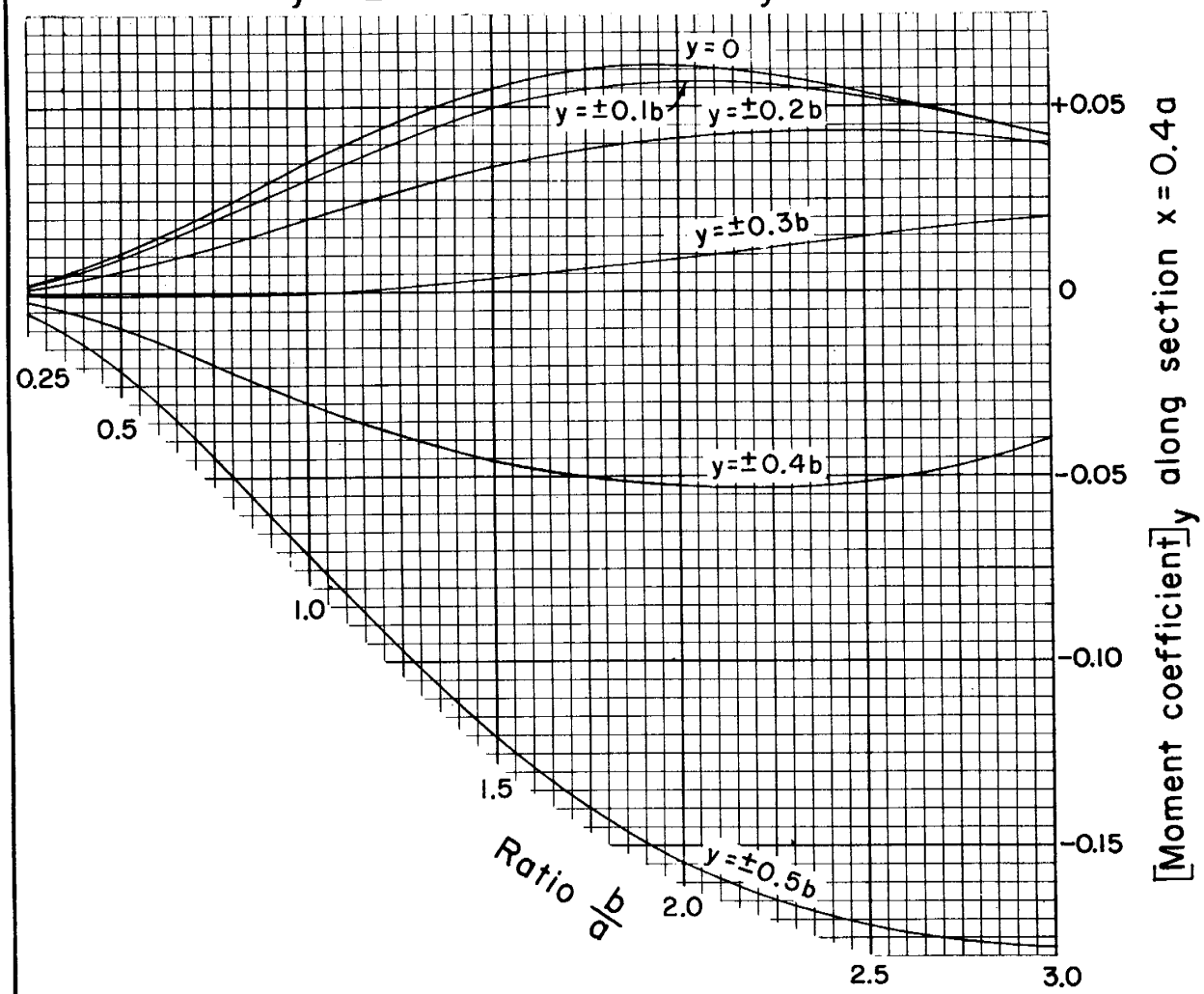
x = 0.2a

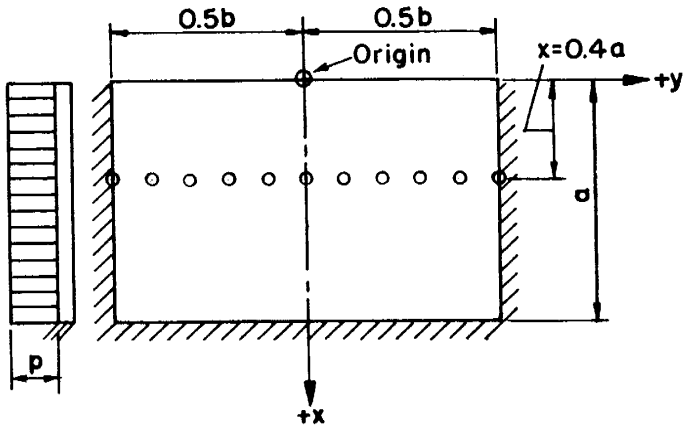
REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 51 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with uniform load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice $x = 0.4a$

Horizontal moment determines tension in horizontal steel

$M_y = [\text{Moment coefficient}]_y \, pa^2$





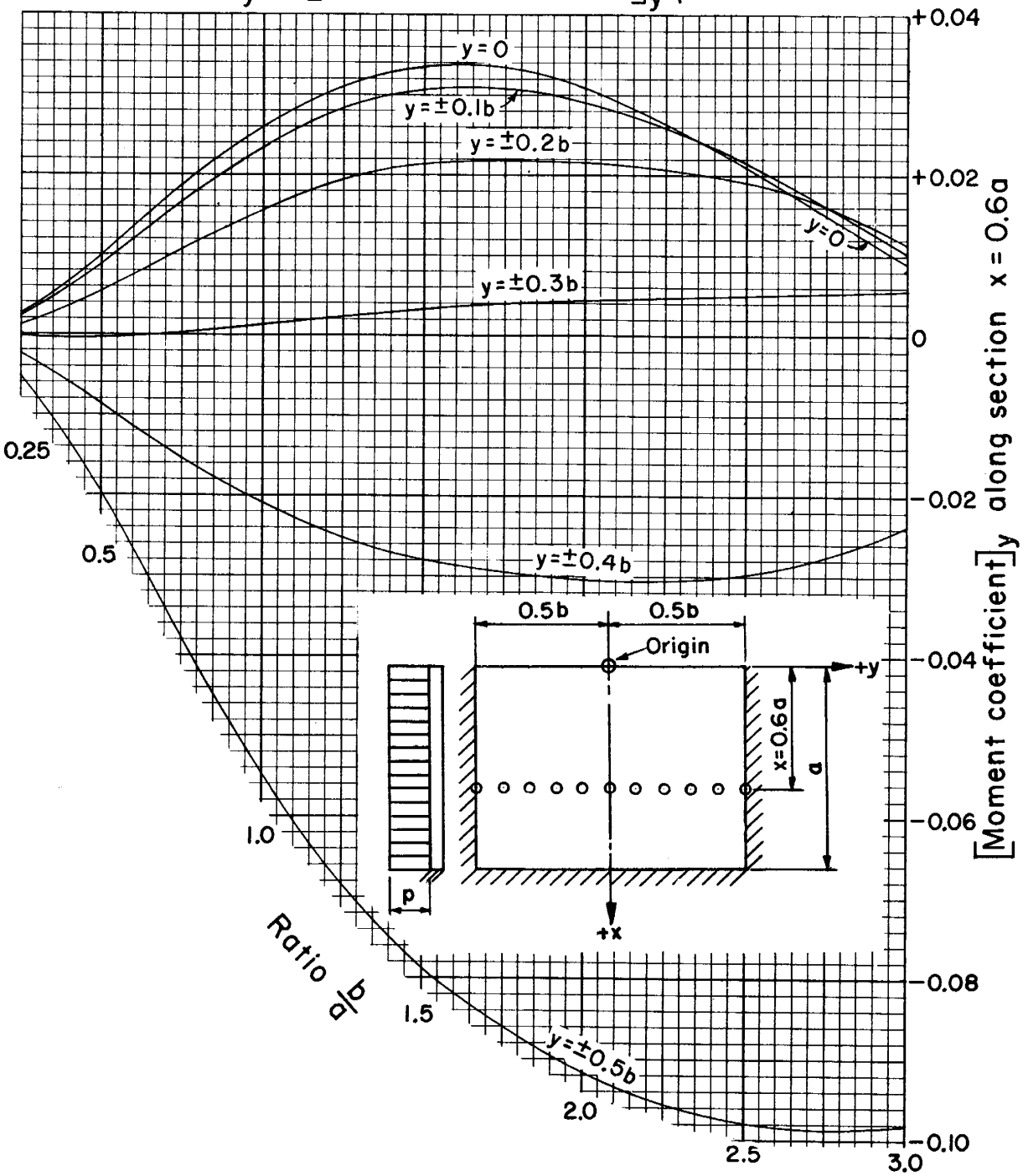
$x = 0.4a$

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 52 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with uniform load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice $x = 0.6a$

Horizontal moment determines tension in horizontal steel

$M_y = [\text{Moment coefficient}]_y \, pa^2$



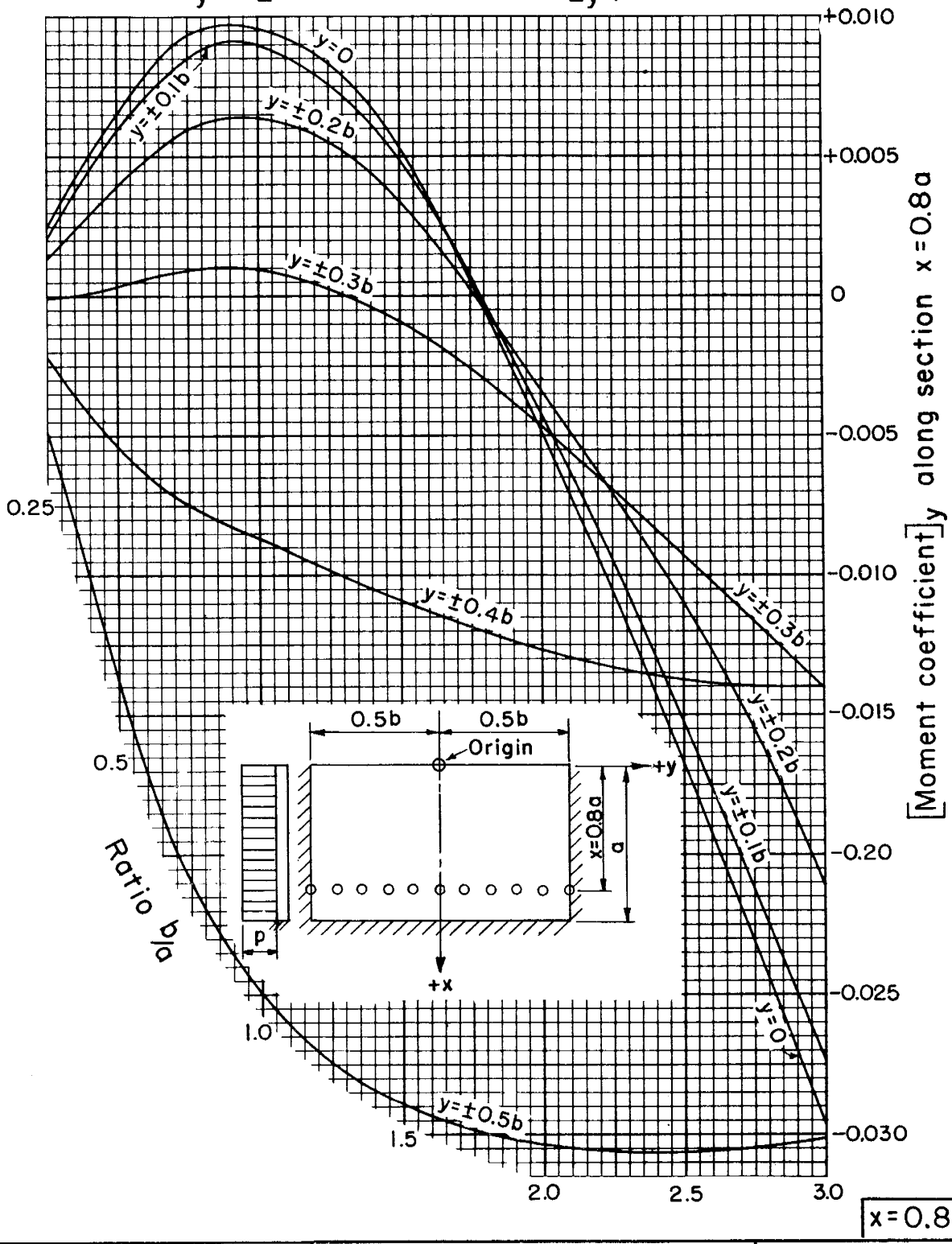
x = 0.6a

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 53 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with uniform load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice $x = 0.8a$

Horizontal moment determines tension in horizontal steel

$M_y = [\text{Moment coefficient}]_y \, pa^2$

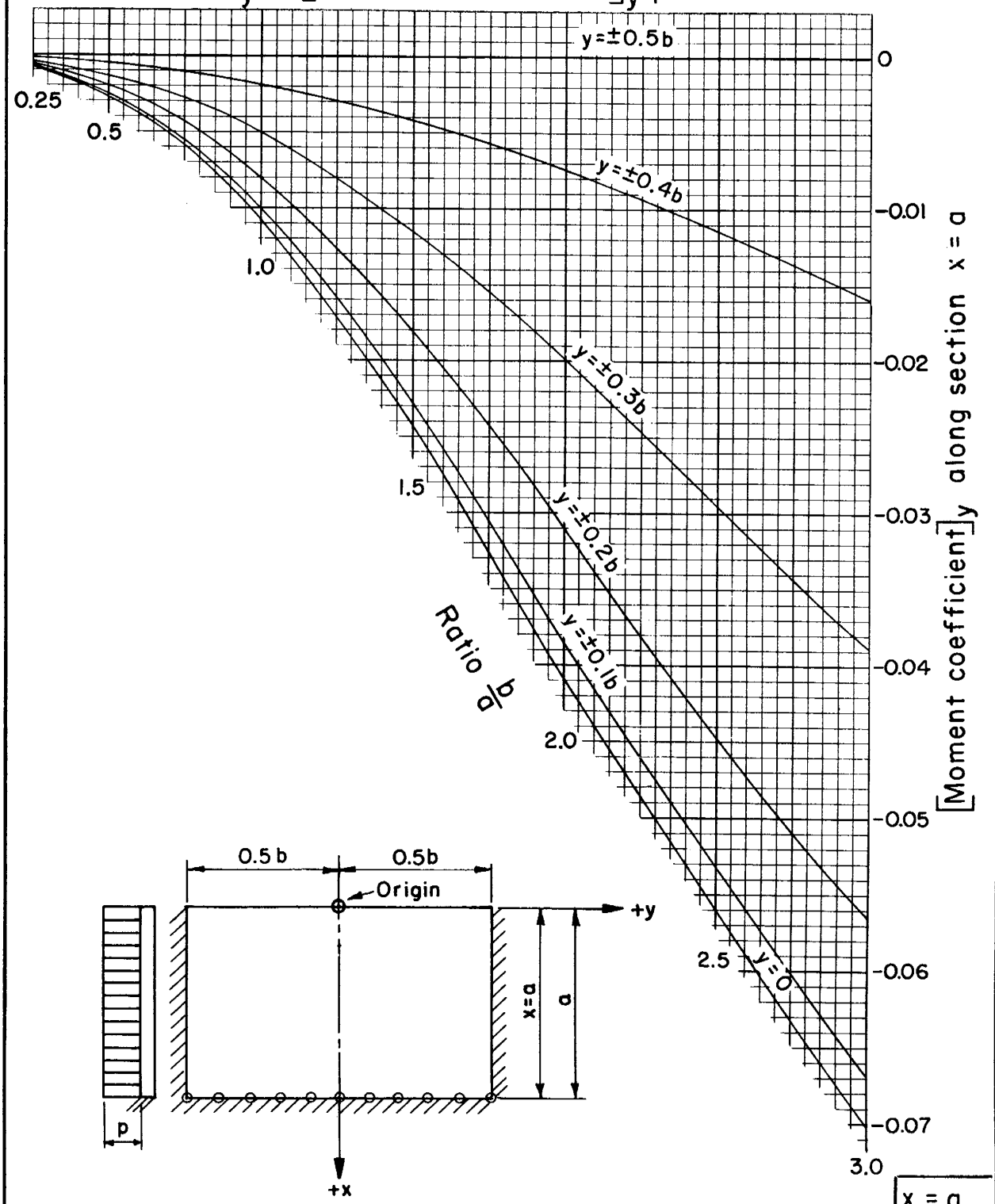


REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 54 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with uniform load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = a

Horizontal moment determines tension in horizontal steel

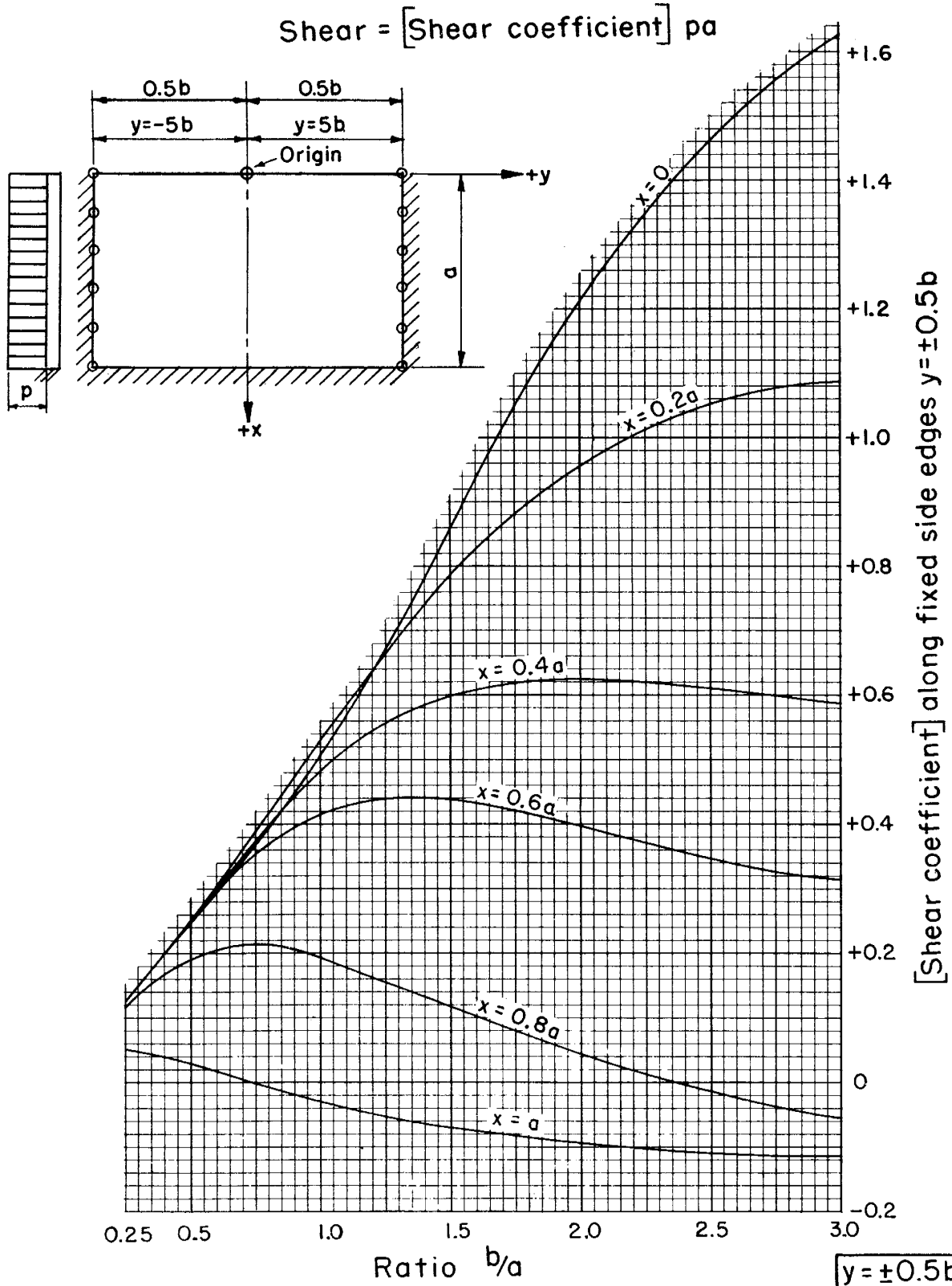
M_y = [Moment coefficient]$_y$ pa^2



x = a

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 55 OF 85
		DATE 8-1-55

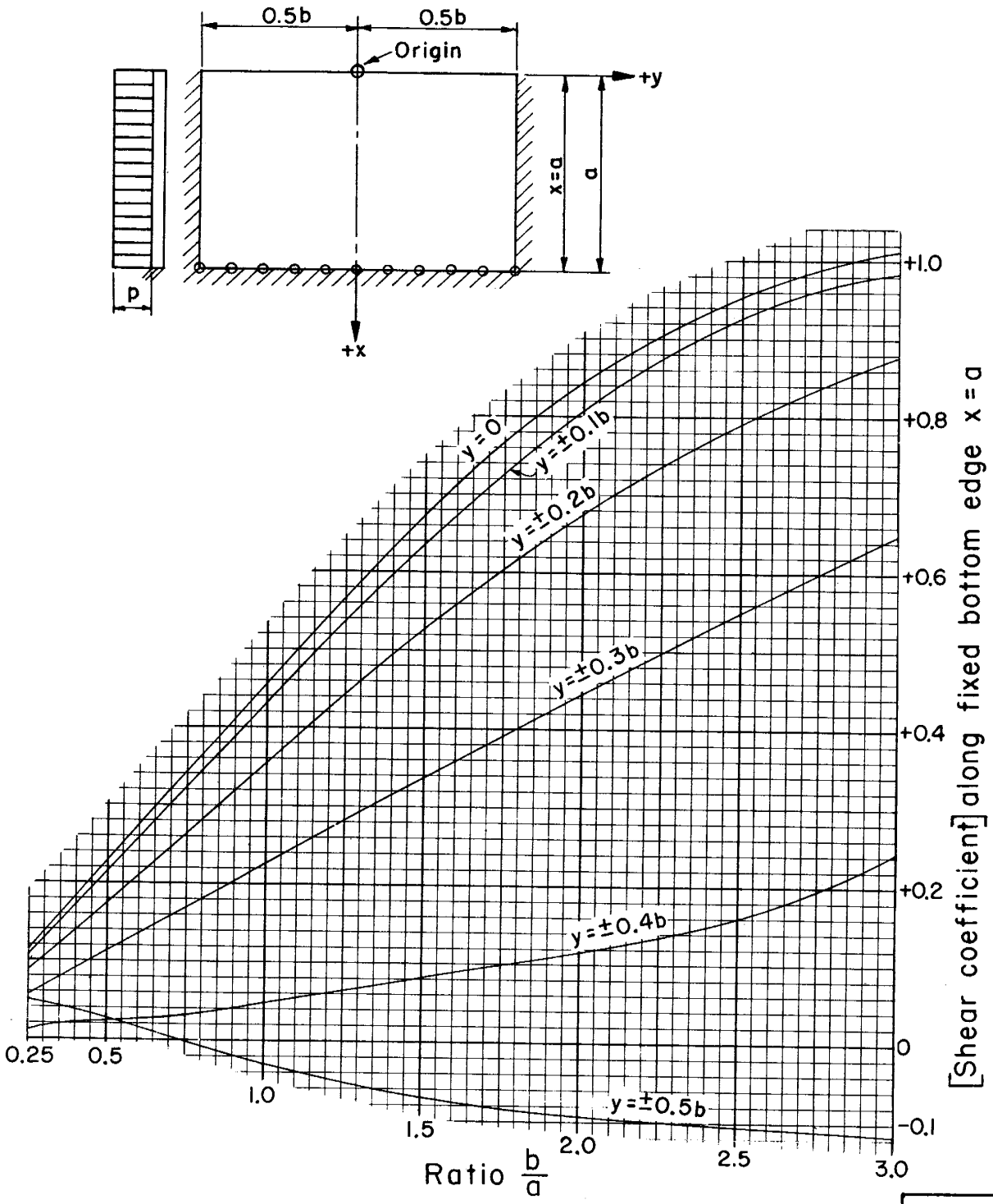
STRUCTURAL DESIGN: Rectangular slabs with uniform load; coefficients for shear at fifth points on fixed side edges $y = \pm 0.5b$



REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 56 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with uniform load; coefficients for shear at tenth points on fixed bottom edge x = a

Shear = [Shear coefficient] pa



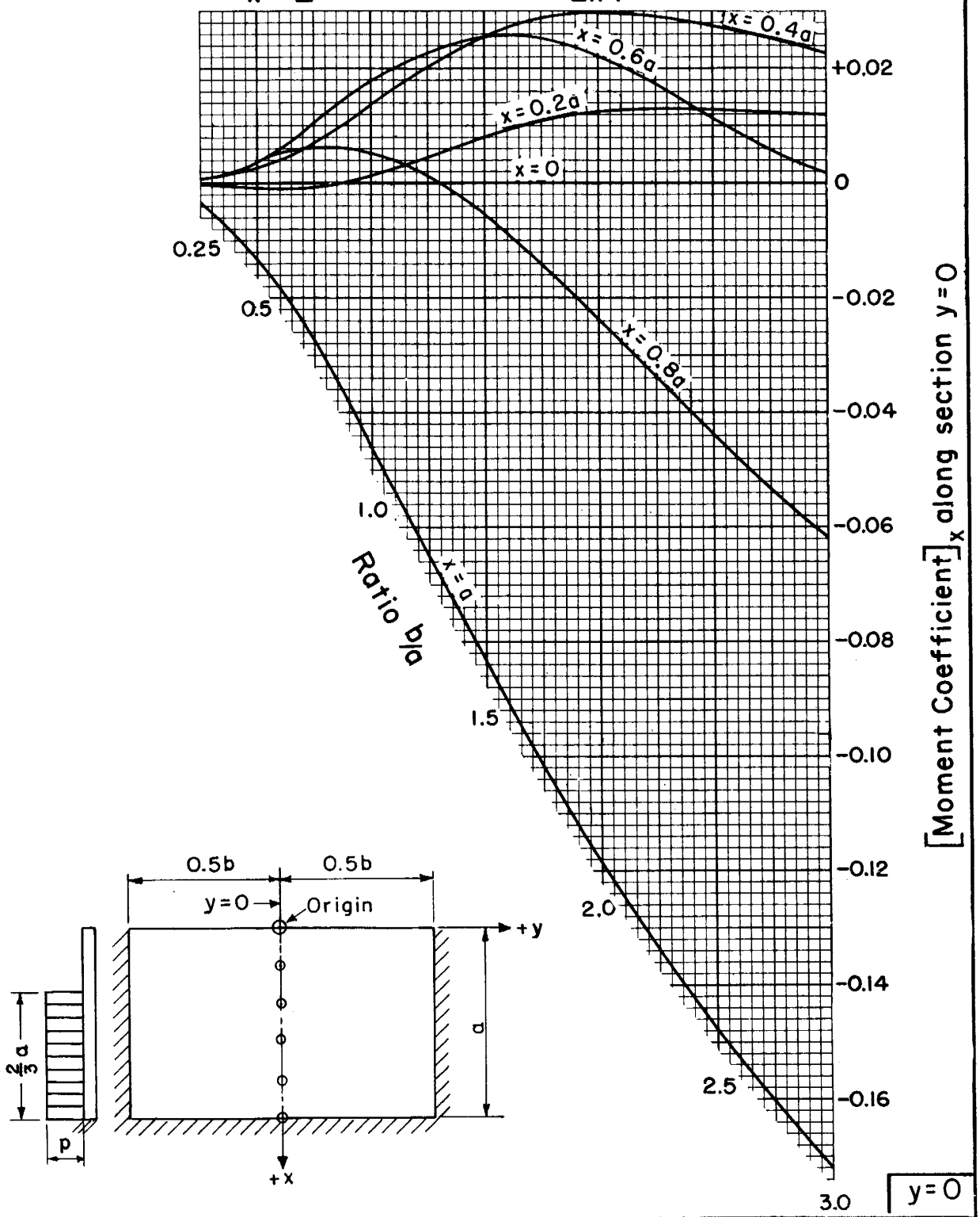
x = a

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 57 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with $\frac{2}{3}$ uniform load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = 0$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \, pa^2$

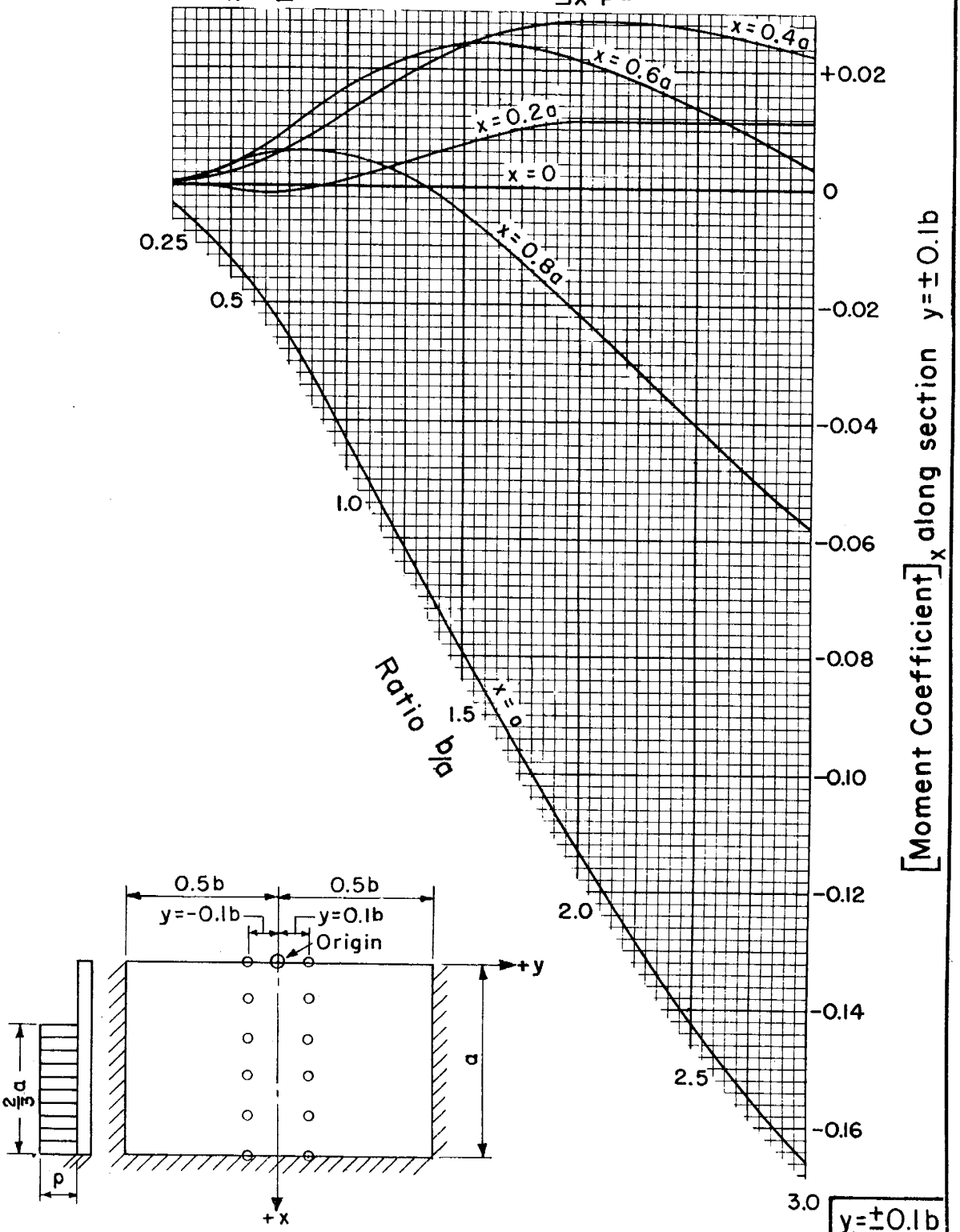


REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 58 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with 2/3 uniform load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.1b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \, pa^2$



REFERENCE

U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
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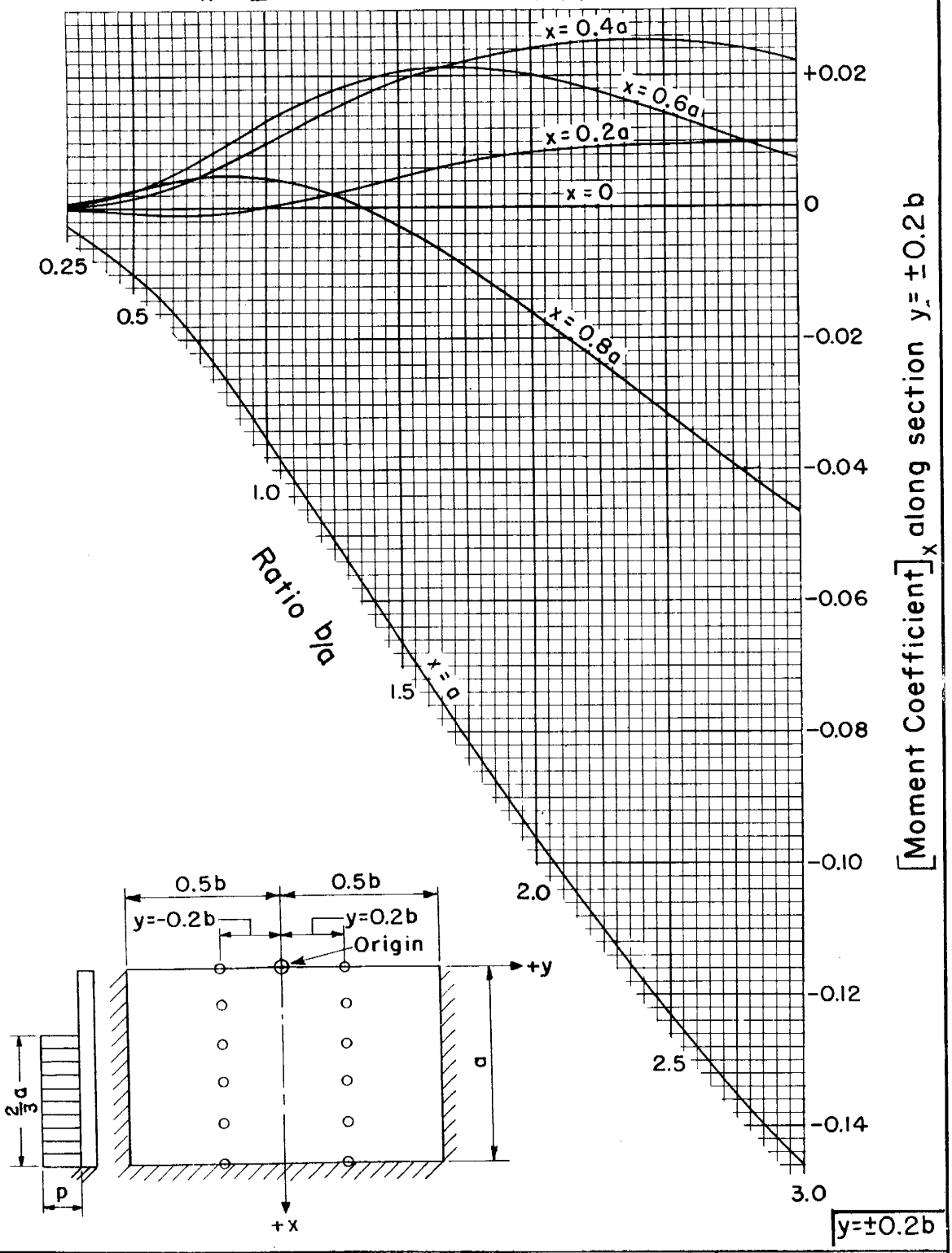
DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with $\frac{2}{3}$ uniform load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.2b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \, pa^2$



REFERENCE

U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954

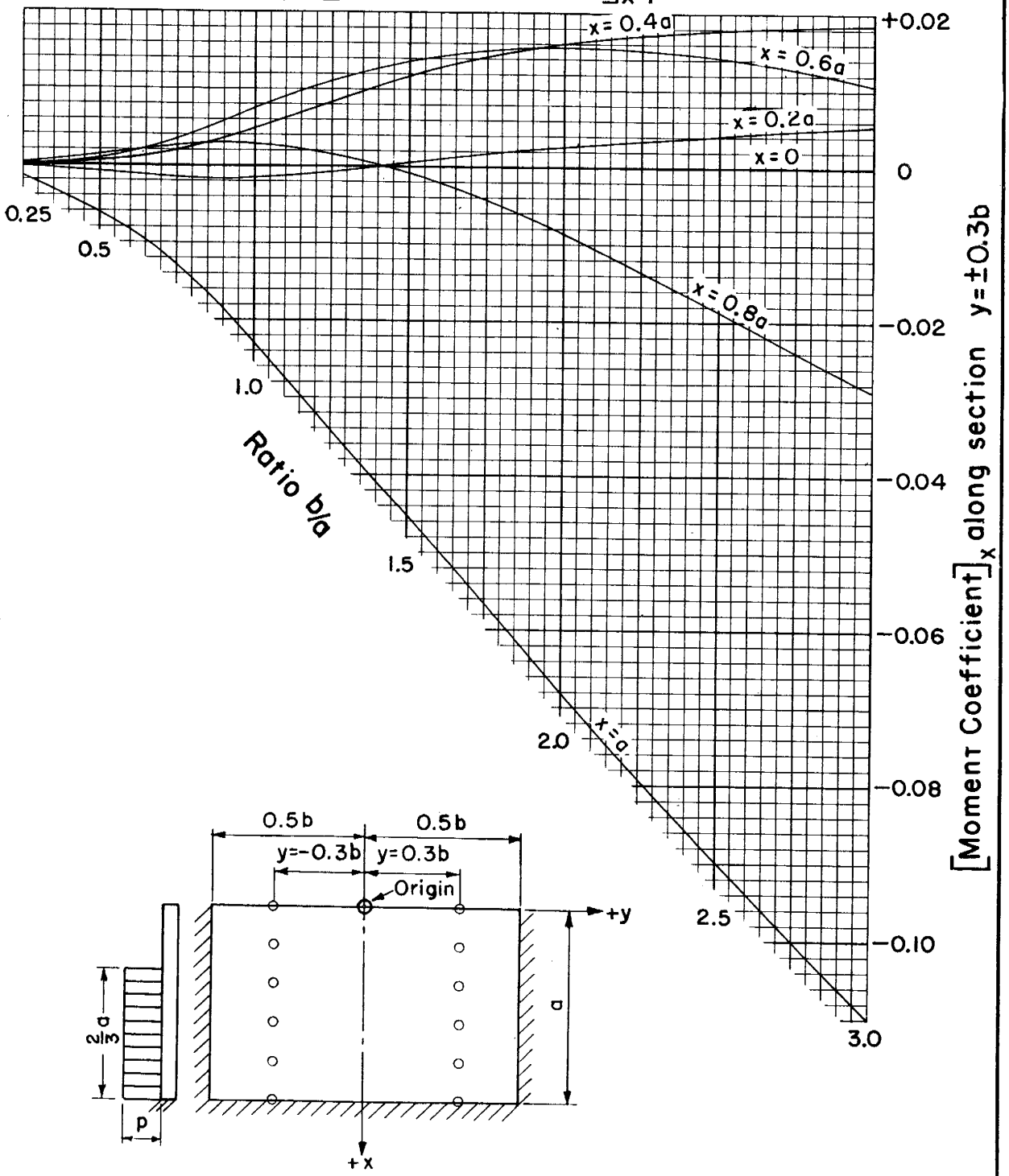
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES-104
SHEET 60 OF 85
DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with $\frac{2}{3}$ uniform load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.3b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \, pa^2$



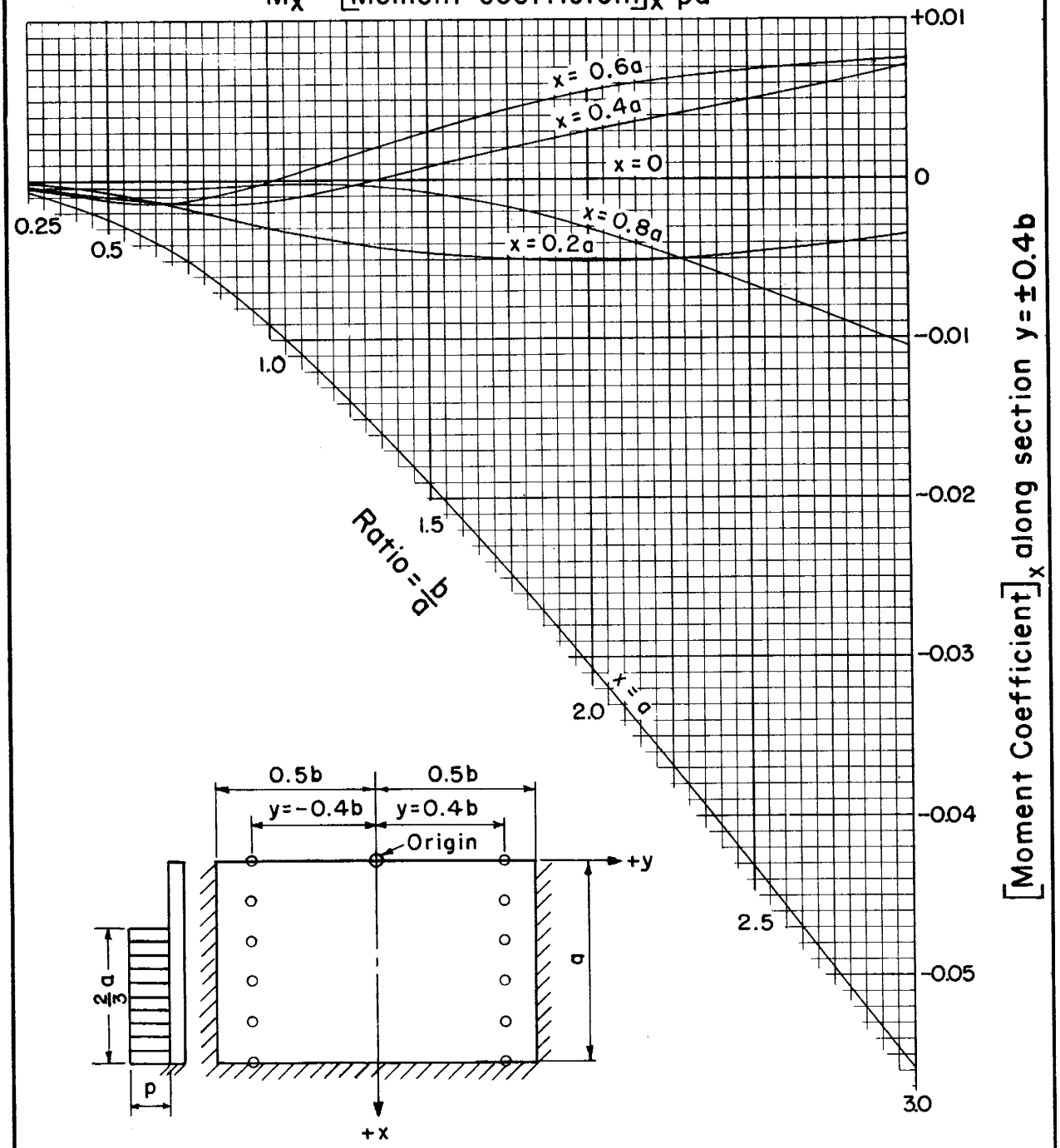
$y = \pm 0.3b$

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 61 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with $\frac{2}{3}$ uniform load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.4b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \, pa^2$



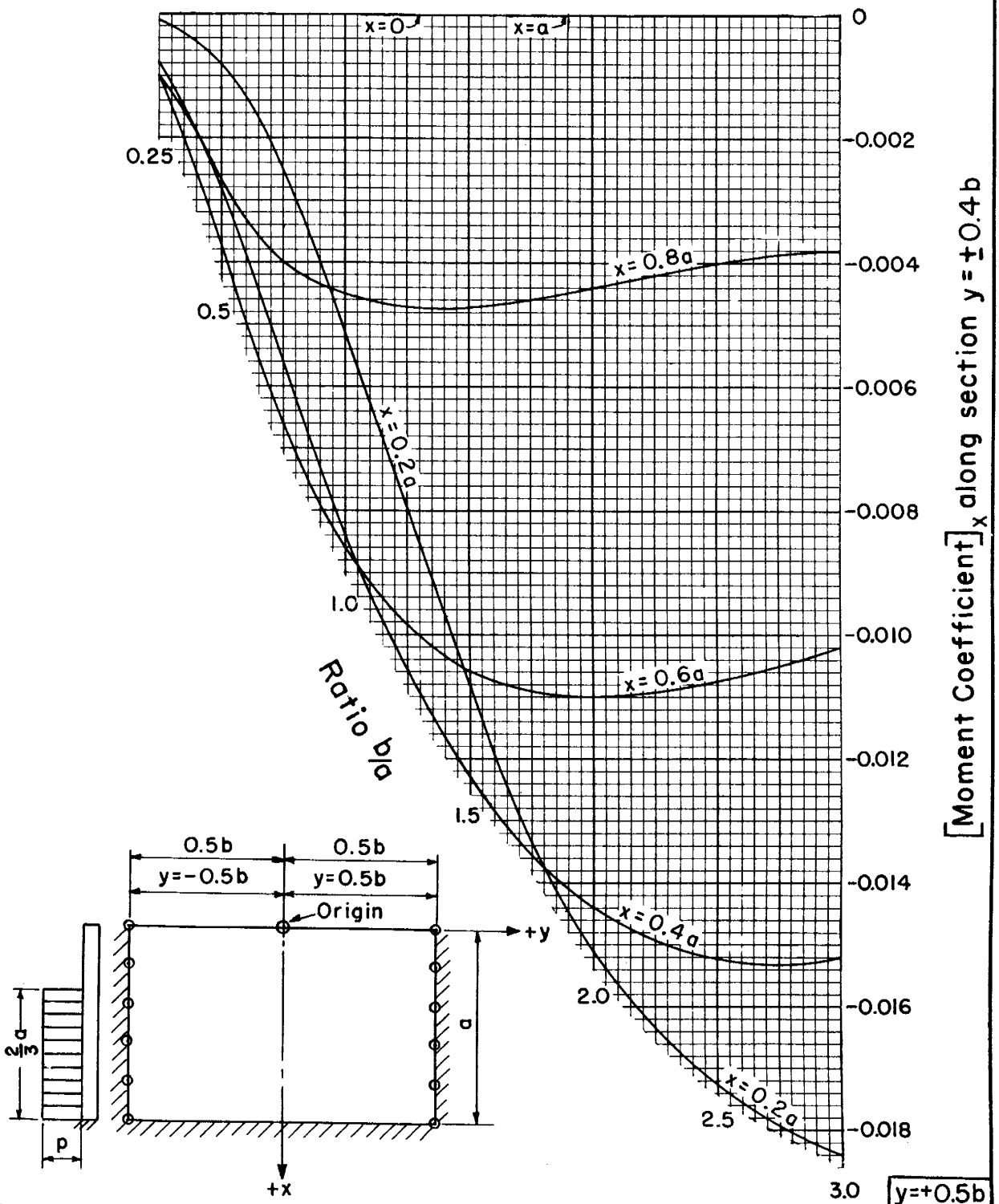
$y = \pm 0.4b$

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 62 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with $\frac{2}{3}$ uniform load; coefficients for vertical moment, M_x, at fifth points on vertical slice $y = \pm 0.5b$

Vertical moment determines tension in vertical steel

$M_x = [\text{Moment coefficient}]_x \, pa^2$

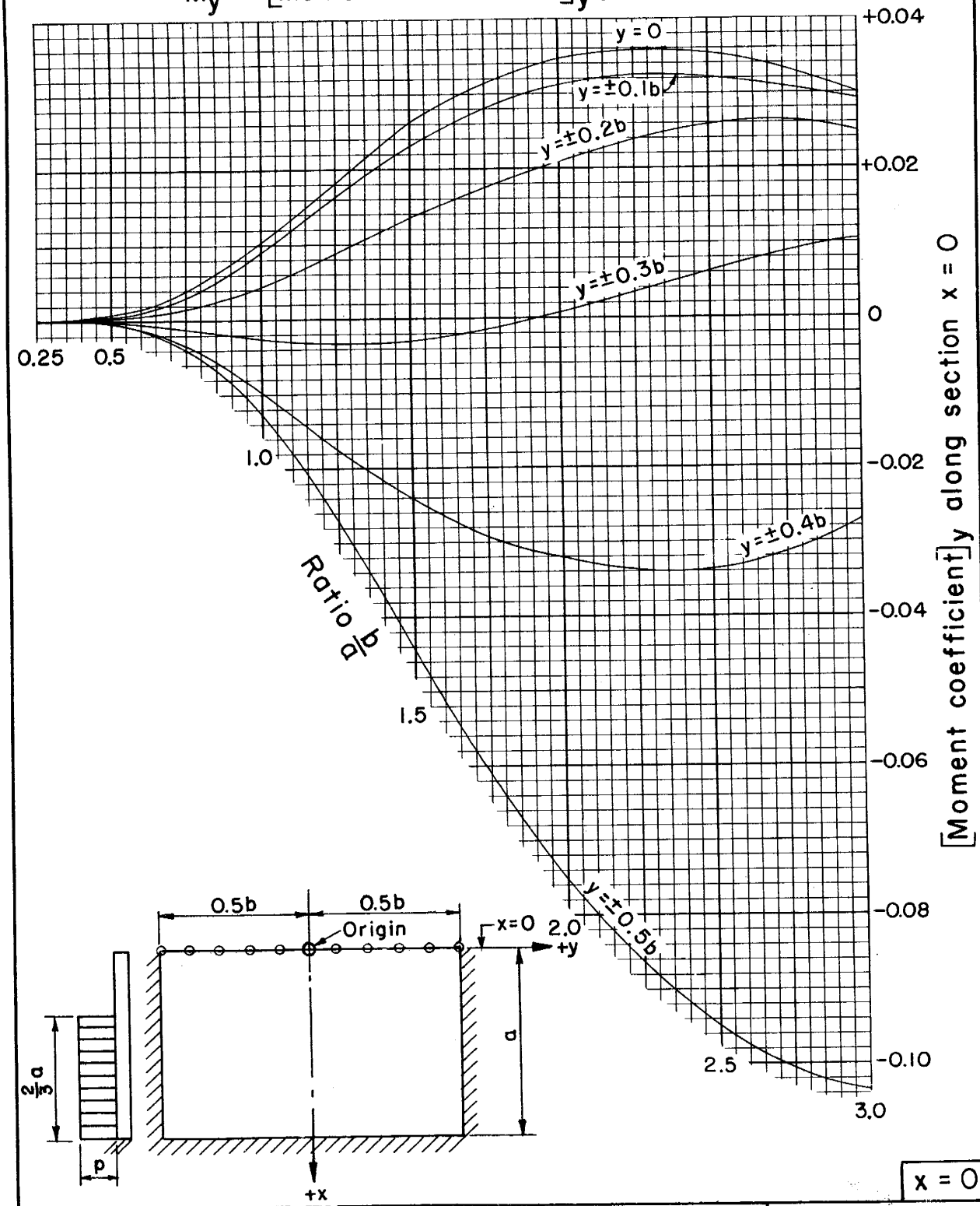


REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 63 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with 2/3 uniform load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = 0

Horizontal moment determines tension in horizontal steel

$$M_y = [\text{Moment coefficient}]_y \, pa^2$$



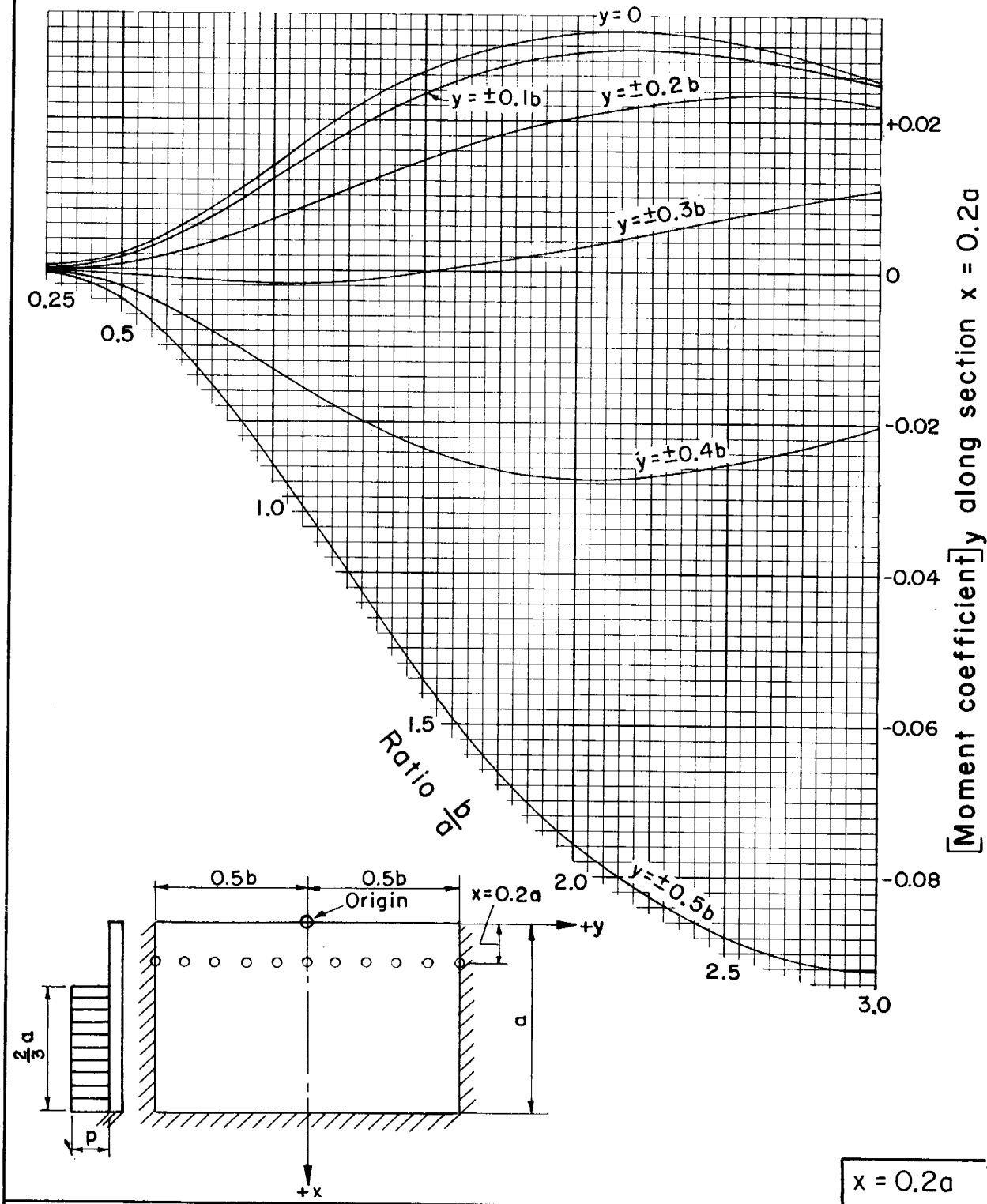
x = 0

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 64 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with $^2/_3$ uniform load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = 0.2a

Horizontal moment determines tension in horizontal steel

$$M_y = [\text{Moment coefficient}]_y \, pa^2$$



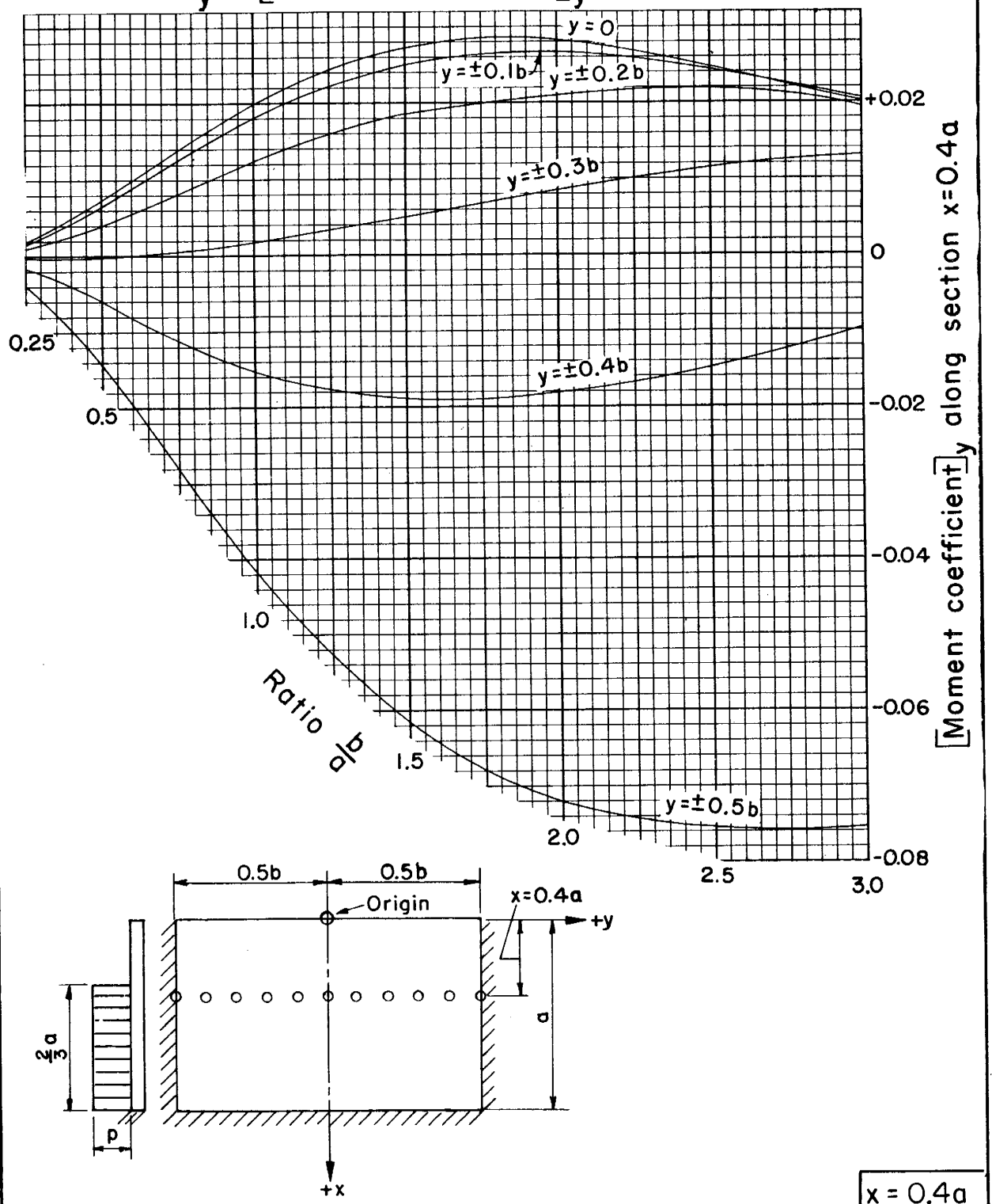
x = 0.2a

REFERENCE		
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - DESIGN SECTION	STANDARD DWG. NO. ES-104 SHEET 65 OF 85 DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with 2/3 uniform load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = 0.4a

Horizontal moment determines tension in horizontal steel

$$M_y = [\text{Moment coefficient}]_y \, pa^2$$



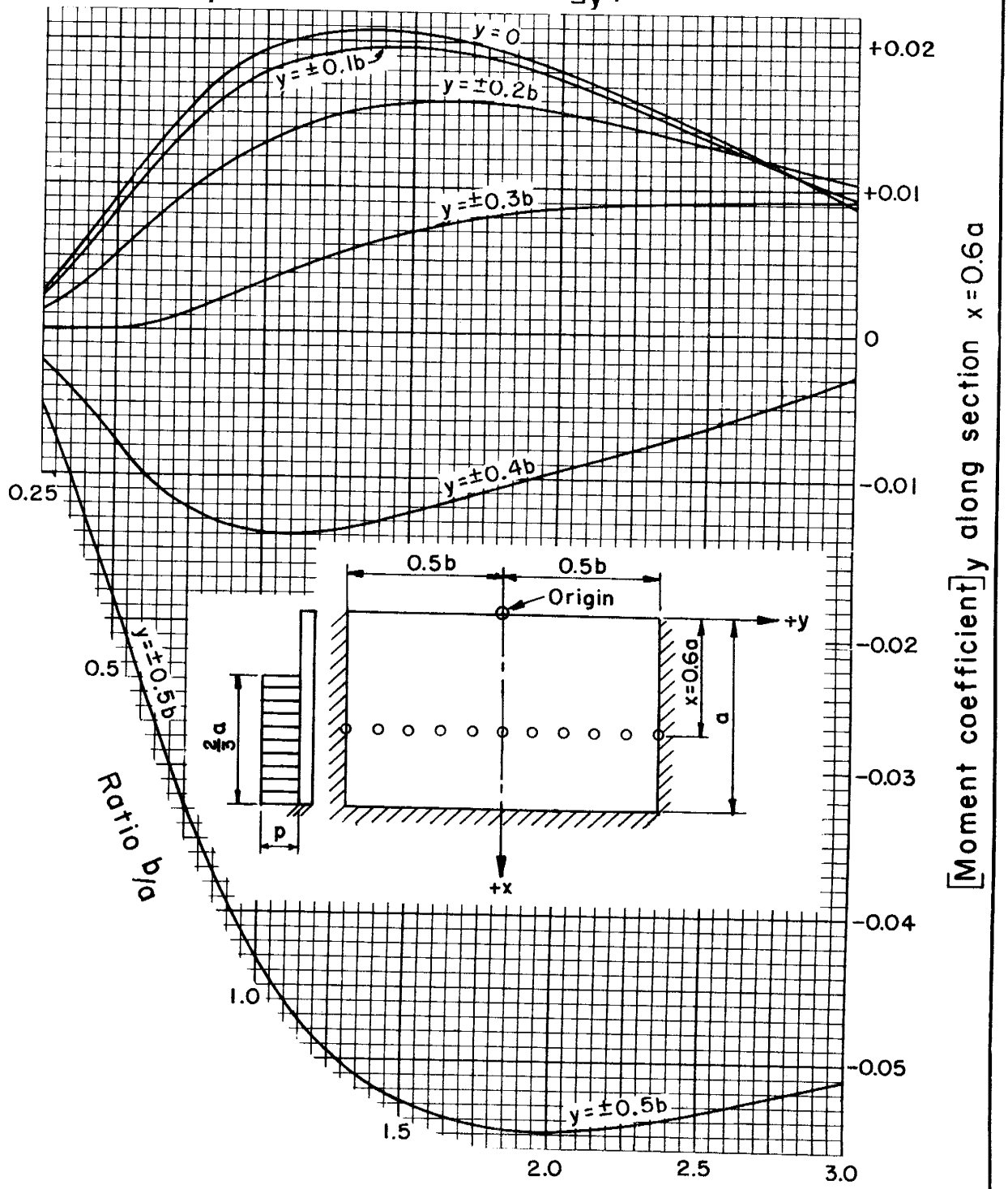
x = 0.4a

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 66 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with 2/3 uniform load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = 0.6a

Horizontal moment determines tension in horizontal steel

M_y = [Moment coefficient]$_y$ pa^2



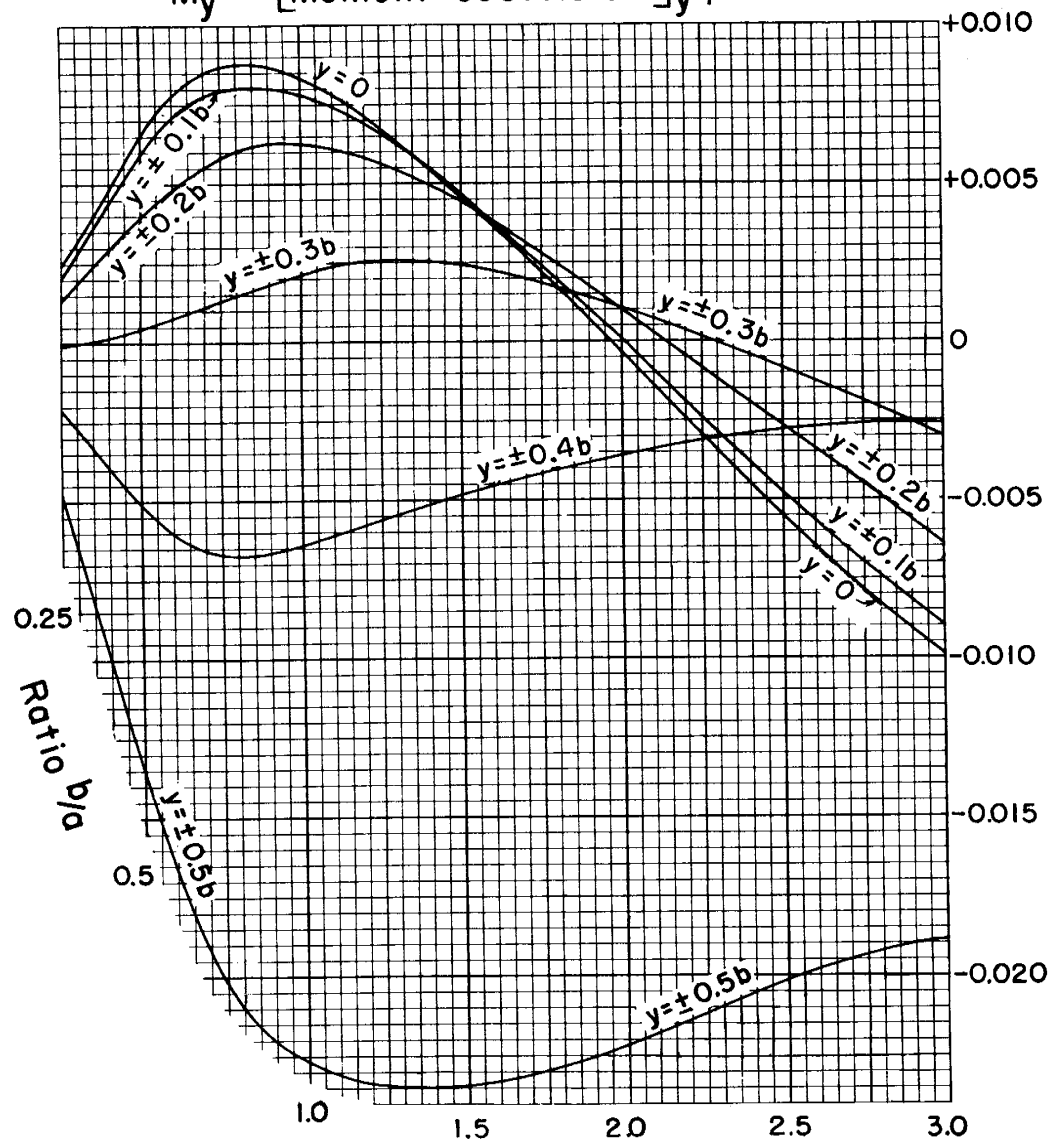
x = 0.6a

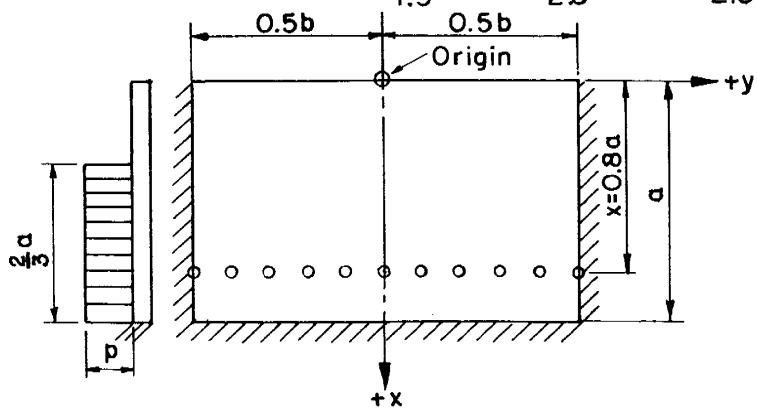
REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 67 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with 2/3 uniform load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = 0.8a

Horizontal moment determines tension in horizontal steel

M_y = [Moment coefficient]$_y$ pa^2





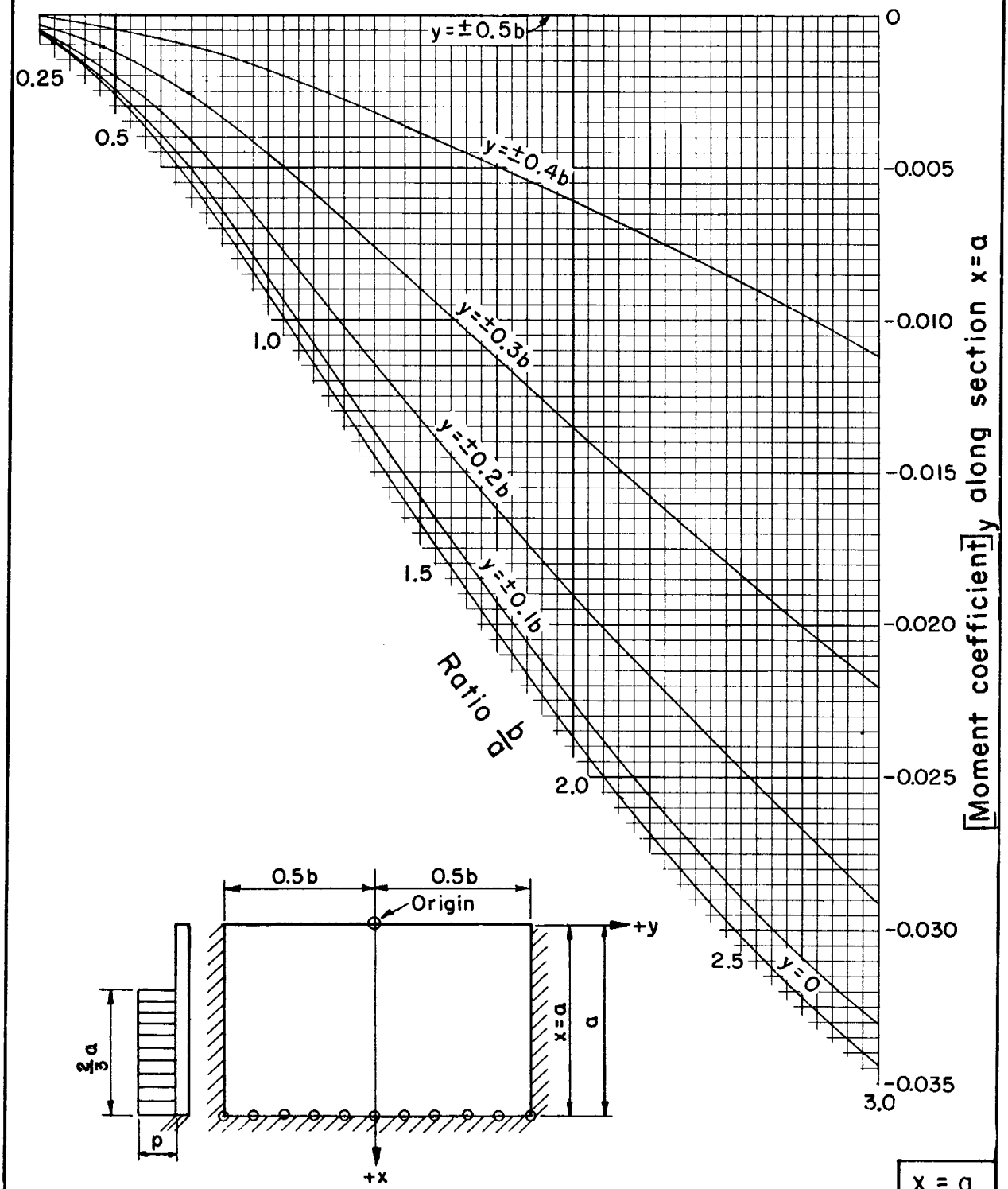
x=0.8a

REFERENCE Bureau of Reclamation photoelastic unit report No. 30, December 1954	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - DESIGN SECTION	STANDARD DWG. NO. ES-104 SHEET 68 OF 85 DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with $^2/_3$ uniform load; coefficients for horizontal moment, M_y, at tenth points on horizontal slice x = a

Horizontal moment determines tension in horizontal steel

$$M_y = [\text{Moment coefficient}]_y \, pa^2$$

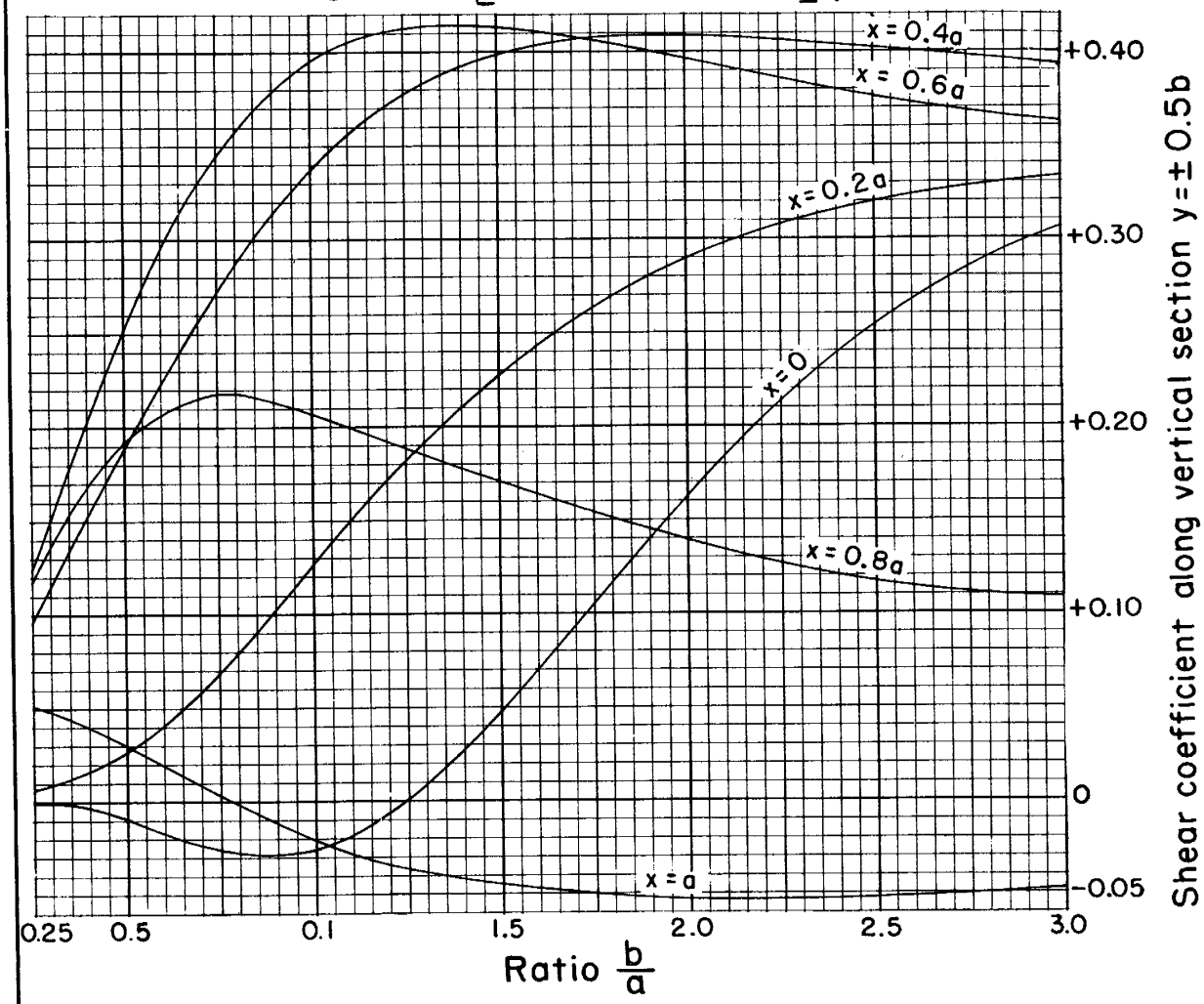


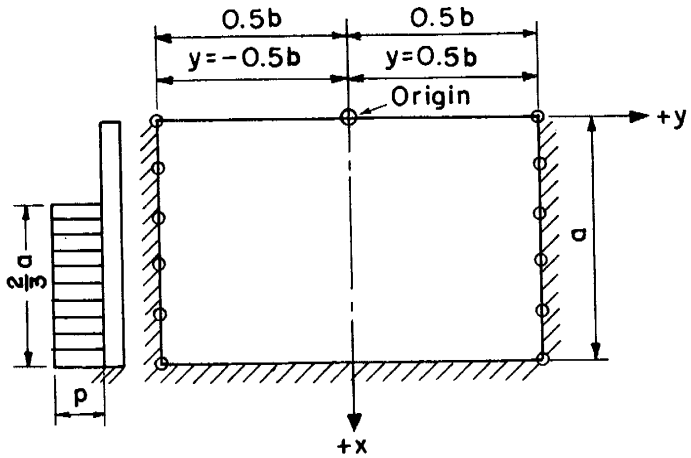
x = a

REFERENCE		
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - DESIGN SECTION	STANDARD DWG. NO. ES-104 SHEET 69 OF 85 DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with 2/3 uniform load; Coefficients for shear at fifth points on fixed side edges $y = \pm 0.5b$

Shear = [Shear coefficient] pa



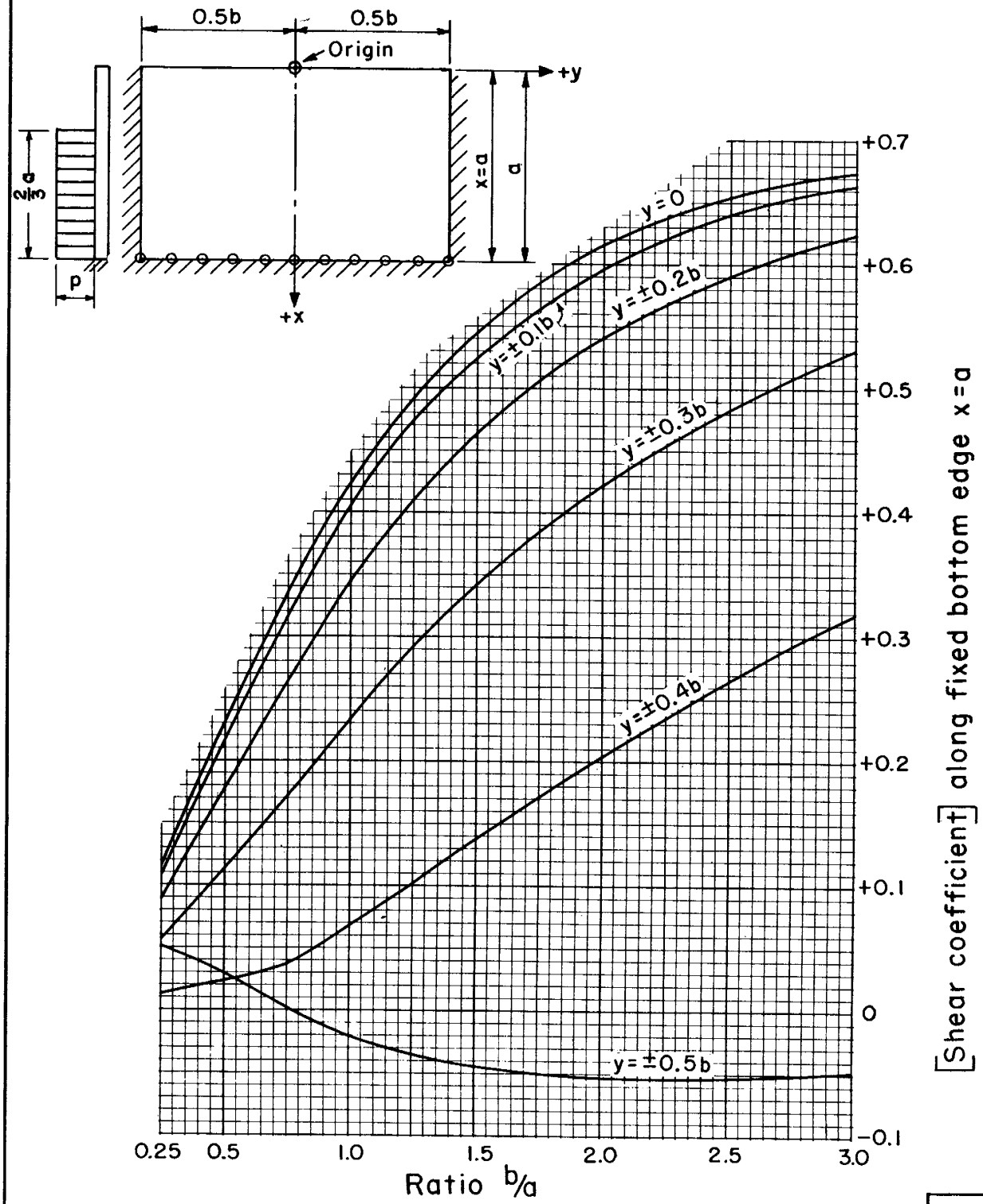


$y = \pm 0.5b$

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954	SOIL CONSERVATION SERVICE	ES-104
	ENGINEERING DIVISION - DESIGN SECTION	SHEET 70 OF 85
		DATE 8-1-55

STRUCTURAL DESIGN: Rectangular slabs with 2/3 uniform load; coefficients for shear at tenth points on fixed bottom edge x = a

Shear = [Shear coefficient] pa



x = a

REFERENCE
U. S. Bureau of Reclamation photoelastic analysis unit report No. 30, December 1954

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
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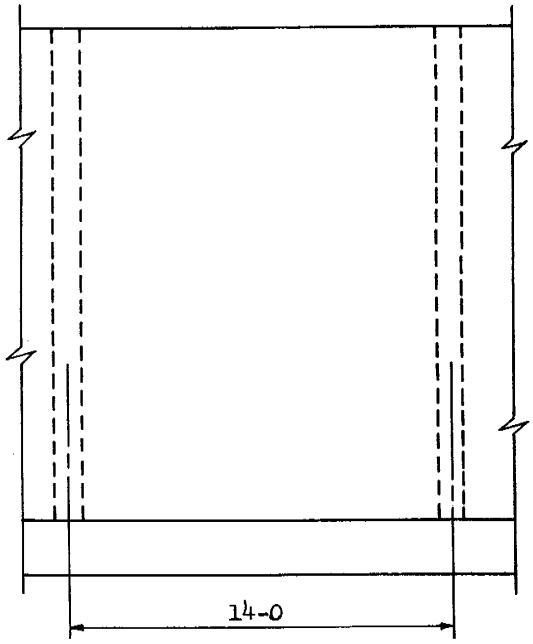
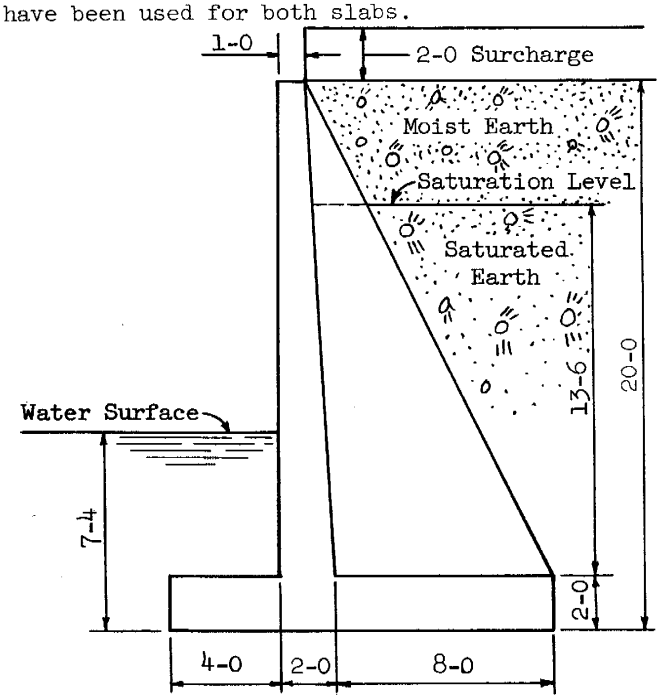
STRUCTURAL DESIGN : DESIGN EXAMPLE ;
Dimensions and design loads

DESIGN EXAMPLE

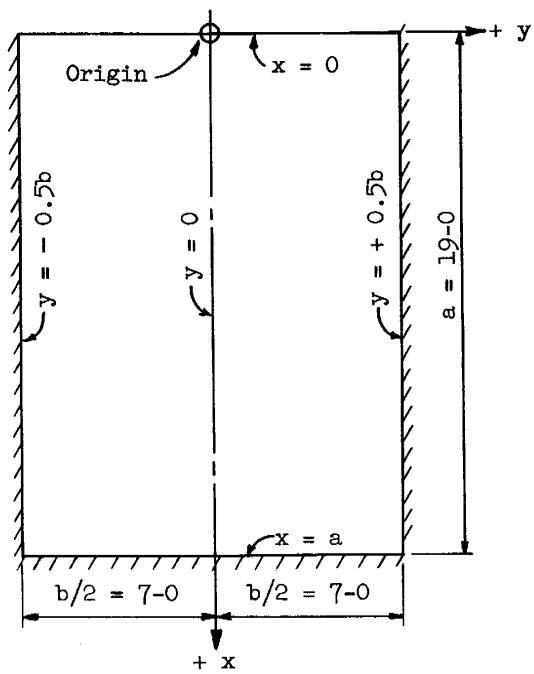
This is strictly an academic example and is only complete insofar as it illustrates the use of the Moment and Shear curves of ES-104. The following figure shows the essential dimensions and possible loads on the interior panel of a counterforted retaining wall. Both the wall slab and the heel slab approximate a plate fixed on three edges and free on the fourth. Center line dimensions have been used for both slabs.



FRONT ELEVATION



SIDE ELEVATION



IDEALIZED WALL SLAB DIMENSIONS

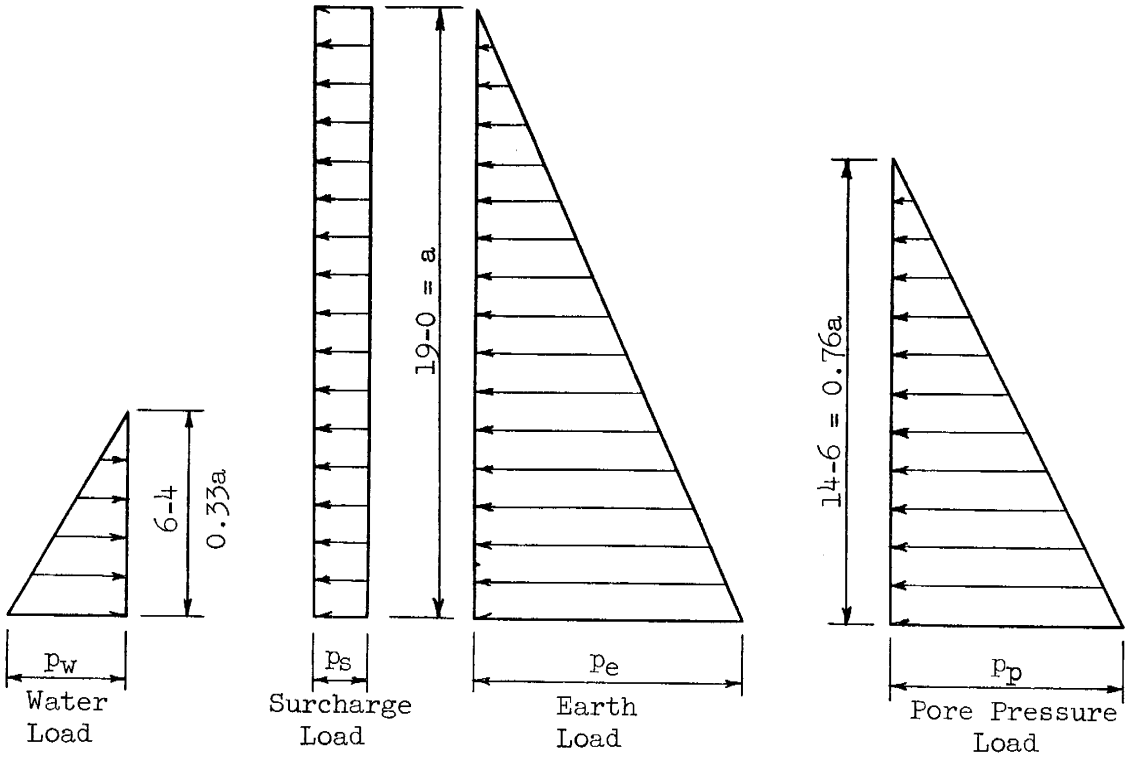
Unit Weights

Concrete	150 lbs/ft^3
Moist Earth	125 lbs/ft^3
Saturated Earth	140 lbs/ft^3
Water	62.4 lbs/ft^3

Equivalent Fluid Weights

Moist Earth	65 lbs/ft^3
Saturated Earth	85 lbs/ft^3

STRUCTURAL DESIGN : DESIGN EXAMPLE ;
Component wall slab loads



COMPONENT WALL SLAB LOADS

$p_w = wh = (62.4)(6.33) = 395 \text{ lbs/ft}^2$

$p_s = wh = (65)(2) = 130 \text{ lbs/ft}^2$

$p_e = wh = (65)(19) = 1235 \text{ lbs/ft}^2$

$p_p = wh = (20)(14.5) = 290 \text{ lbs/ft}^2$

$p_w a = (0.395)(19) = 7.5 \text{ kips/ft}$

$p_s a = (0.130)(19) = 2.5 \text{ kips/ft}$

$p_e a = (1.235)(19) = 23.5 \text{ kips/ft}$

$p_p a = (0.290)(19) = 5.5 \text{ kips/ft}$

$p_w a^2 = (0.395)(19)^2 = 142.6 \text{ ft kips/ft}$

$p_s a^2 = (0.130)(19)^2 = 46.9 \text{ ft kips/ft}$

$p_e a^2 = (1.235)(19)^2 = 445.8 \text{ ft kips/ft}$

$p_p a^2 = (0.290)(19)^2 = 104.7 \text{ ft kips/ft}$

$\frac{b}{a} = \frac{14}{19} = 0.737$

STRUCTURAL DESIGN : DESIGN EXAMPLE ;
Vertical moments (M_x) in wall slab

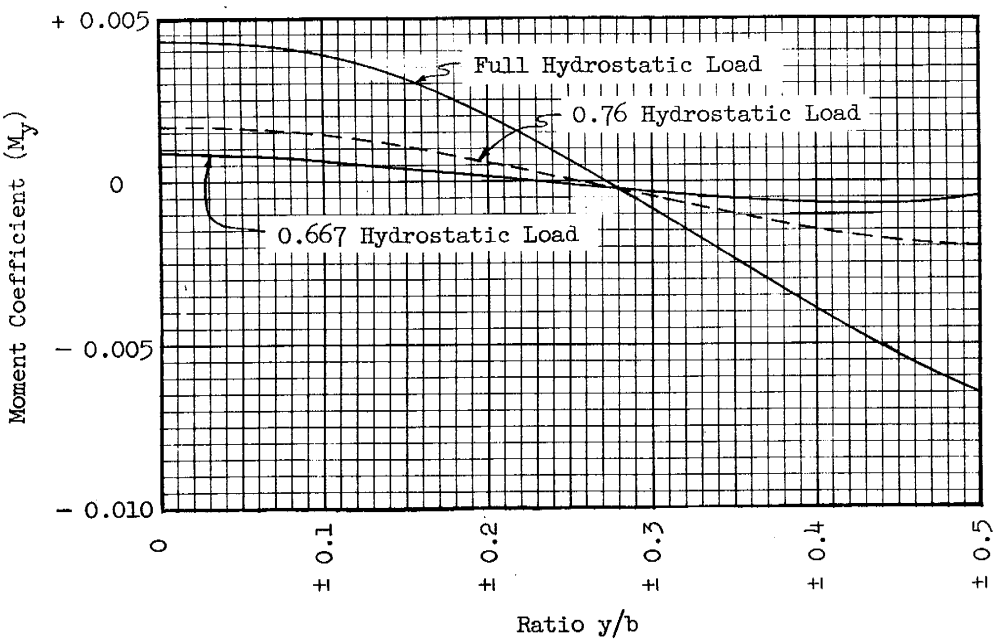
Values		Moment Coefficients				Moments (ft kips)				Total Moment (ft kips)
$pn^2 \rightarrow$		− 142.6	46.9	445.8	104.7					
$\pm \frac{y}{b}$	$\frac{x}{a}$	p_w	p_s	p_e	p_p	M_w	M_s	M_e	M_p	
0	0	0	0	0	0	0	0	0	0	0
0	0.2	− 0.0001	+ 0.0045	+ 0.0008	− 0.0004	+ 0.01	+ 0.21	+ 0.36	− 0.04	+ 0.54
0	0.4	− 0.0002	+ 0.0065	+ 0.0030	+ 0.0013	+ 0.03	+ 0.30	+ 1.34	+ 0.14	+ 1.81
0	0.6	+ 0.0005	+ 0.0085	+ 0.0060	+ 0.0049	− 0.07	+ 0.40	+ 2.67	+ 0.51	+ 3.51
0	0.8	+ 0.0028	+ 0.0048	+ 0.0053	+ 0.0051	− 0.40	+ 0.23	+ 2.36	+ 0.53	+ 2.72
0	1.0	− 0.0078	− 0.0280	− 0.0200	− 0.0170	+ 1.11	− 1.31	− 8.92	− 1.78	− 10.90
± 0.1	0	0	0	0	0	0	0	0	0	0
± 0.1	0.2	− 0.0001	+ 0.0040	+ 0.0007	− 0.0003	+ 0.01	+ 0.19	+ 0.31	− 0.03	+ 0.48
± 0.1	0.4	− 0.0002	+ 0.0055	+ 0.0026	+ 0.0013	+ 0.03	+ 0.26	+ 1.16	+ 0.14	+ 1.59
± 0.1	0.6	+ 0.0005	+ 0.0080	+ 0.0058	+ 0.0045	− 0.07	+ 0.38	+ 2.59	+ 0.47	+ 3.37
± 0.1	0.8	+ 0.0026	+ 0.0045	+ 0.0048	+ 0.0047	− 0.37	+ 0.21	+ 2.14	+ 0.49	+ 2.47
± 0.1	1.0	− 0.0075	− 0.0251	− 0.0185	− 0.0160	+ 1.07	− 1.18	− 8.25	− 1.68	− 10.04
± 0.2	0	0	0	0	0	0	0	0	0	0
± 0.2	0.2	− 0.0001	+ 0.0020	+ 0.0001	− 0.0004	+ 0.01	+ 0.10	+ 0.04	− 0.04	+ 0.11
± 0.2	0.4	− 0.0002	+ 0.0030	+ 0.0019	+ 0.0008	+ 0.03	+ 0.14	+ 0.85	+ 0.08	+ 1.10
± 0.2	0.6	+ 0.0003	+ 0.0060	+ 0.0040	+ 0.0033	− 0.04	+ 0.28	+ 1.78	+ 0.35	+ 2.37
± 0.2	0.8	+ 0.0022	+ 0.0030	+ 0.0038	+ 0.0038	− 0.31	+ 0.14	+ 1.69	+ 0.40	+ 1.92
± 0.2	1.0	− 0.0063	− 0.0200	− 0.0150	− 0.0130	+ 0.90	− 0.94	− 6.69	− 1.36	− 8.09
± 0.3	0	0	0	0	0	0	0	0	0	0
± 0.3	0.2	− 0.0001	− 0.0005	− 0.0003	− 0.0004	+ 0.01	− 0.02	− 0.13	− 0.04	− 0.18
± 0.3	0.4	− 0.0002	+ 0.0010	+ 0.0005	+ 0.0001	+ 0.03	+ 0.05	+ 0.22	+ 0.01	+ 0.31
± 0.3	0.6	0	+ 0.0021	+ 0.0019	+ 0.0016	0	+ 0.10	+ 0.85	+ 0.17	+ 1.12
± 0.3	0.8	+ 0.0014	+ 0.0019	+ 0.0021	+ 0.0022	− 0.20	+ 0.09	+ 0.94	+ 0.23	+ 1.06
± 0.3	1.0	− 0.0044	− 0.0130	− 0.0095	− 0.0085	+ 0.63	− 0.61	− 4.24	− 0.89	− 5.11
± 0.4	0	0	0	0	0	0	0	0	0	0
± 0.4	0.2	− 0.0001	− 0.0041	− 0.0012	− 0.0007	+ 0.01	− 0.19	− 0.53	− 0.07	− 0.78
± 0.4	0.4	− 0.0002	− 0.0036	− 0.0013	− 0.0009	+ 0.03	− 0.17	− 0.58	− 0.09	− 0.81
± 0.4	0.6	− 0.0003	− 0.0026	− 0.0009	− 0.0006	+ 0.04	− 0.12	− 0.40	− 0.06	− 0.54
± 0.4	0.8	+ 0.0003	− 0.0010	− 0.0001	+ 0.0001	− 0.04	− 0.05	− 0.04	+ 0.01	− 0.12
± 0.4	1.0	− 0.0020	− 0.0050	− 0.0038	− 0.0035	+ 0.29	− 0.23	− 1.69	− 0.37	− 2.00
± 0.5	0	0	0	0	0	0	0	0	0	0
± 0.5	0.2	− 0.0001	− 0.0092	− 0.0022	− 0.0010	+ 0.01	− 0.43	− 0.98	− 0.10	− 1.50
± 0.5	0.4	− 0.0002	− 0.0087	− 0.0034	− 0.0021	+ 0.03	− 0.41	− 1.52	− 0.22	− 2.12
± 0.5	0.6	− 0.0006	− 0.0072	− 0.0042	− 0.0031	+ 0.09	− 0.34	− 1.87	− 0.32	− 2.44
± 0.5	0.8	− 0.0010	− 0.0040	− 0.0028	− 0.0024	+ 0.14	− 0.19	− 1.25	− 0.25	− 1.55
± 0.5	1.0	0	0	0	0	0	0	0	0	0

STRUCTURAL DESIGN ; DESIGN EXAMPLE ;
Horizontal moments (M_y) in wall slab

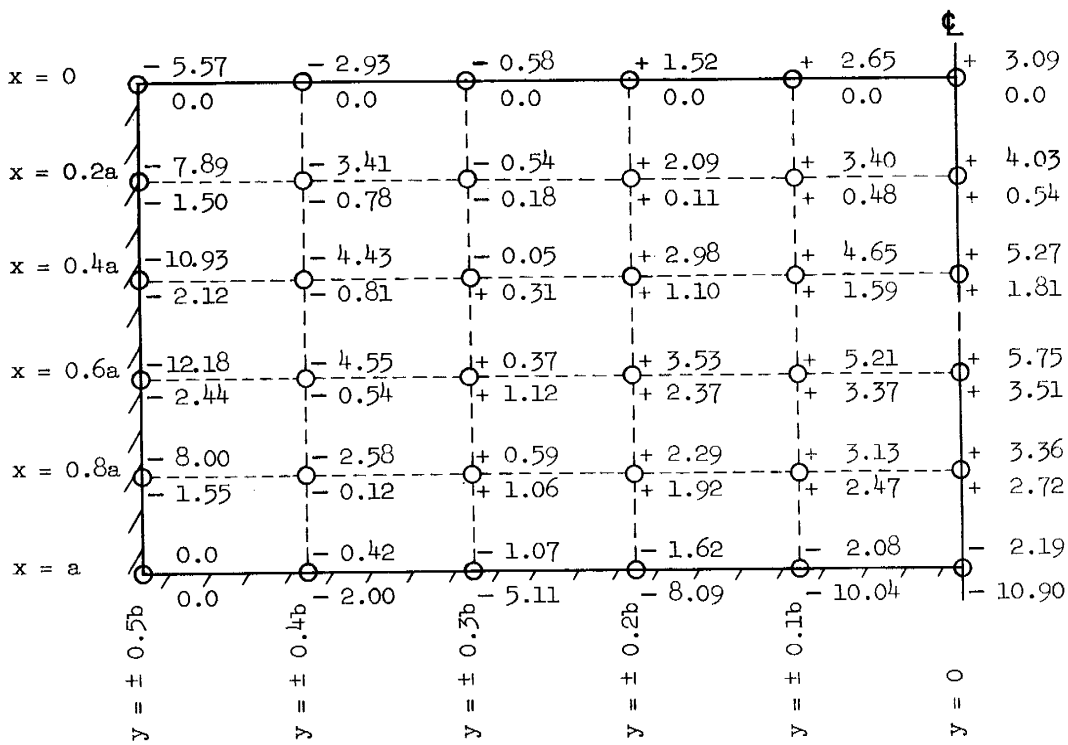
Values		Moment Coefficients				Moments (ft kips)				Total Moment (ft kips)
$pa^2 \rightarrow$		− 142.6	46.9	445.8	104.7					
$\frac{x}{a}$	$\pm \frac{y}{b}$	p_w	p_s	p_e	p_p	M_w	M_s	M_e	M_p	
0	0	+ 0.0001	+ 0.0240	+ 0.0040	+ 0.0018	− 0.01	+ 1.13	+ 1.78	+ 0.19	+ 3.09
0	± 0.1	0	+ 0.0200	+ 0.0035	+ 0.0014	0	+ 0.94	+ 1.56	+ 0.15	+ 2.65
0	± 0.2	0	+ 0.0120	+ 0.0020	+ 0.0007	0	+ 0.56	+ 0.89	+ 0.07	+ 1.52
0	± 0.3	0	− 0.0020	− 0.0010	− 0.0004	0	− 0.09	− 0.45	− 0.04	− 0.58
0	± 0.4	0	− 0.0210	− 0.0040	− 0.0016	0	− 0.98	− 1.78	− 0.17	− 2.93
0	± 0.5	0	− 0.0470	− 0.0070	− 0.0024	0	− 2.20	− 3.12	− 0.25	− 5.57
0.2	± 0	+ 0.0002	+ 0.0230	+ 0.0060	+ 0.0030	− 0.03	+ 1.08	+ 2.67	+ 0.31	+ 4.03
0.2	± 0.1	+ 0.0002	+ 0.0200	+ 0.0050	+ 0.0025	− 0.03	+ 0.94	+ 2.23	+ 0.26	+ 3.40
0.2	± 0.2	+ 0.0001	+ 0.0130	+ 0.0030	+ 0.0014	− 0.01	+ 0.61	+ 1.34	+ 0.15	+ 2.09
0.2	± 0.3	− 0.0001	− 0.0010	− 0.0010	− 0.0005	+ 0.01	− 0.05	− 0.45	− 0.05	− 0.54
0.2	± 0.4	− 0.0002	− 0.0200	− 0.0050	− 0.0026	+ 0.03	− 0.94	− 2.23	− 0.27	− 3.41
0.2	± 0.5	− 0.0003	− 0.0480	− 0.0115	− 0.0053	+ 0.04	− 2.25	− 5.13	− 0.55	− 7.89
0.4	0	+ 0.0006	+ 0.0220	+ 0.0085	+ 0.0052	− 0.09	+ 1.03	+ 3.78	+ 0.54	+ 5.27
0.4	± 0.1	+ 0.0005	+ 0.0190	+ 0.0075	+ 0.0047	− 0.07	+ 0.89	+ 3.34	+ 0.49	+ 4.65
0.4	± 0.2	+ 0.0002	+ 0.0120	+ 0.0048	+ 0.0030	− 0.03	+ 0.56	+ 2.14	+ 0.31	+ 2.98
0.4	± 0.3	− 0.0001	− 0.0010	0	− 0.0001	+ 0.01	− 0.05	0	− 0.01	− 0.05
0.4	± 0.4	− 0.0006	− 0.0180	− 0.0072	− 0.0045	+ 0.09	− 0.84	− 3.21	− 0.47	− 4.43
0.4	± 0.5	− 0.0010	− 0.0450	− 0.0176	− 0.0106	+ 0.14	− 2.11	− 7.85	− 1.11	− 10.93
0.6	0	+ 0.0013	+ 0.0183	+ 0.0097	+ 0.0073	− 0.19	+ 0.86	+ 4.32	+ 0.76	+ 5.75
0.6	± 0.1	+ 0.0012	+ 0.0163	+ 0.0088	+ 0.0067	− 0.17	+ 0.76	+ 3.92	+ 0.70	+ 5.21
0.6	± 0.2	+ 0.0007	+ 0.0104	+ 0.0060	+ 0.0045	− 0.10	+ 0.49	+ 2.67	+ 0.47	+ 3.53
0.6	± 0.3	0	0	+ 0.0007	+ 0.0006	0	0	+ 0.31	+ 0.06	+ 0.37
0.6	± 0.4	− 0.0012	− 0.0150	− 0.0077	− 0.0056	+ 0.17	− 0.70	− 3.43	− 0.59	− 4.55
0.6	± 0.5	− 0.0028	− 0.0370	− 0.0207	− 0.0154	+ 0.40	− 1.74	− 9.23	− 1.61	− 12.18
0.8	0	+ 0.0019	+ 0.0093	+ 0.0060	+ 0.0050	− 0.27	+ 0.44	+ 2.67	+ 0.52	+ 3.36
0.8	± 0.1	+ 0.0018	+ 0.0084	+ 0.0056	+ 0.0048	− 0.26	+ 0.39	+ 2.50	+ 0.50	+ 3.13
0.8	± 0.2	+ 0.0016	+ 0.0058	+ 0.0042	+ 0.0036	− 0.23	+ 0.27	+ 1.87	+ 0.38	+ 2.29
0.8	± 0.3	+ 0.0008	+ 0.0008	+ 0.0012	+ 0.0012	− 0.11	+ 0.04	+ 0.53	+ 0.13	+ 0.59
0.8	± 0.4	− 0.0011	− 0.0074	− 0.0045	− 0.0036	+ 0.16	− 0.35	− 2.01	− 0.38	− 2.58
0.8	± 0.5	− 0.0051	− 0.0210	− 0.0145	− 0.0123	+ 0.73	− 0.98	− 6.46	− 1.29	− 8.00
1	0	− 0.0016	− 0.0059	− 0.0040	− 0.0034	+ 0.23	− 0.28	− 1.78	− 0.36	− 2.19
1	± 0.1	− 0.0015	− 0.0055	− 0.0038	− 0.0032	+ 0.21	− 0.26	− 1.69	− 0.34	− 2.08
1	± 0.2	− 0.0013	− 0.0043	− 0.0030	− 0.0026	+ 0.19	− 0.20	− 1.34	− 0.27	− 1.62
1	± 0.3	− 0.0009	− 0.0027	− 0.0020	− 0.0017	+ 0.13	− 0.13	− 0.89	− 0.18	− 1.07
1	± 0.4	− 0.0004	− 0.0010	− 0.0008	− 0.0007	+ 0.06	− 0.05	− 0.36	− 0.07	− 0.42
1	± 0.5	0	0	0	0	0	0	0	0	0

STRUCTURAL DESIGN : DESIGN EXAMPLE ;

Example of interpolation, horizontal and vertical moments in wall slab



EXAMPLE OF INTERPOLATION OF MOMENT COEFFICIENT (M_y)
FOR 0.76 HYDROSTATIC LOAD ALONG SECTION x = 0

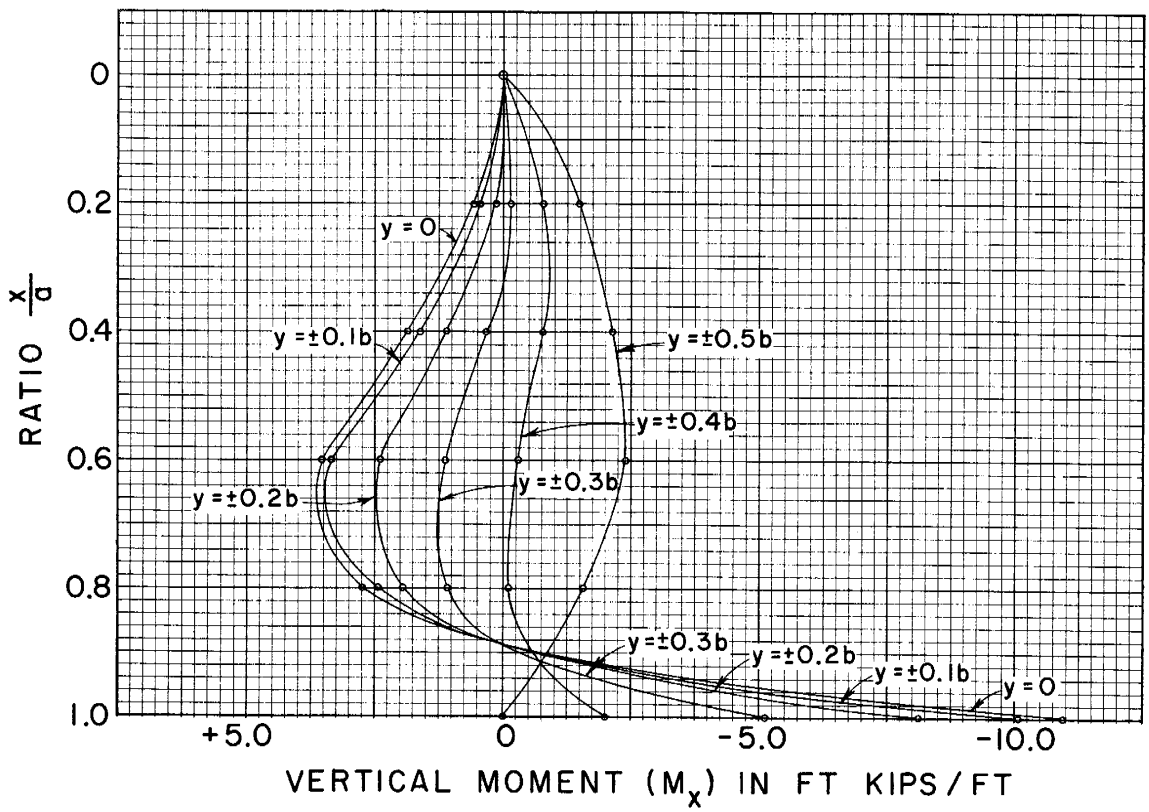


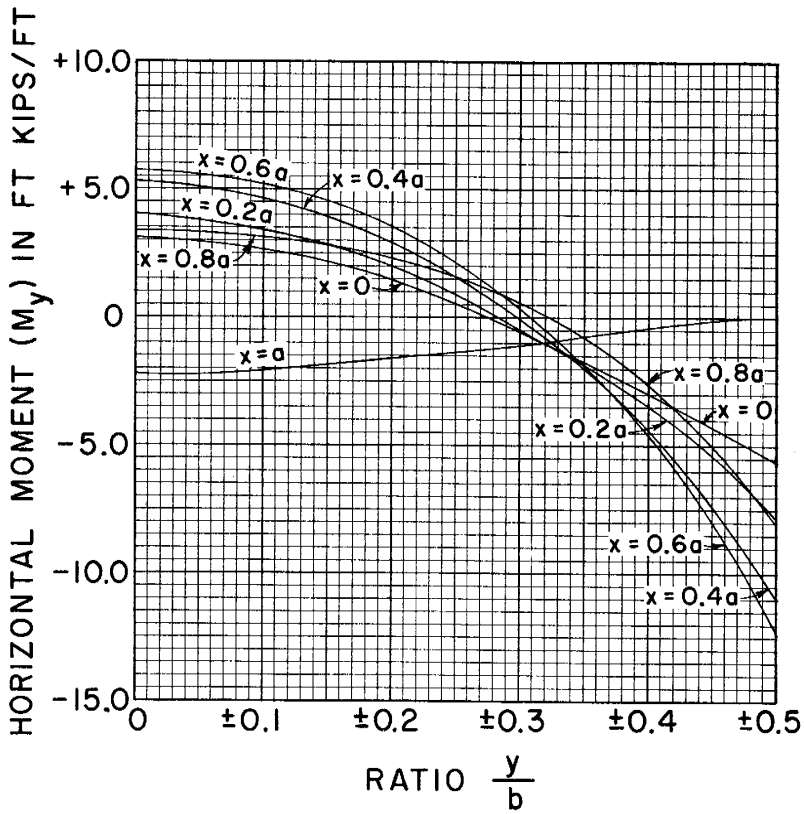
HORIZONTAL AND VERTICAL MOMENTS IN ft kips/ft IN WALL SLAB

Horizontal Moment
Vertical Moment

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - DESIGN SECTION	STANDARD DWG. NO. ES- 104 SHEET 76 OF 85 DATE 4 - 13 - 56

STRUCTURAL DESIGN : DESIGN EXAMPLE ;
Vertical and horizontal moments in wall slab





REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - DESIGN SECTION	STANDARD DWG. NO. ES-104 SHEET 77 OF 85 DATE 4 13 56

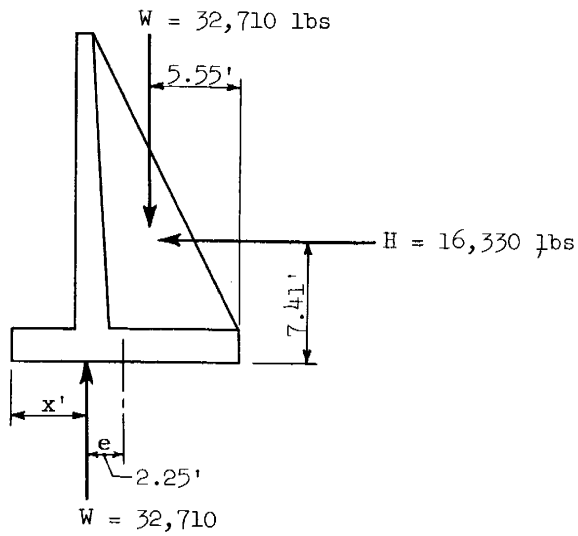
STRUCTURAL DESIGN : DESIGN EXAMPLE ;
Shears in wall slab

Values		Shear Coefficients				Shear (kips/ft)				Total Shear
pa →		− 7.5	2.5	23.5	5.5					
$\frac{x}{a}$	$\pm \frac{y}{b}$	p_w	p_s	p_e	p_p	s_w	s_s	s_e	s_p	$\frac{kips}{ft}$
0	± 0.5	− 0.0021	+ 0.3650	+ 0.0190	− 0.0020	+ 0.02	+ 0.91	+ 0.45	− 0.01	+ 1.37
0.2	± 0.5	+ 0.0001	+ 0.3900	+ 0.0850	+ 0.0305	0.0	+ 0.98	+ 2.00	+ 0.17	+ 3.15
0.4	± 0.5	+ 0.0035	+ 0.3700	+ 0.1500	+ 0.0773	− 0.03	+ 0.93	+ 3.53	+ 0.43	+ 4.86
0.6	± 0.5	+ 0.0215	+ 0.3500	+ 0.2100	+ 0.1570	− 0.16	+ 0.88	+ 4.94	+ 0.86	+ 6.52
0.8	± 0.5	+ 0.0780	+ 0.2150	+ 0.1690	+ 0.1515	− 0.59	+ 0.54	+ 3.97	+ 0.83	+ 4.75
1.0	± 0.5	+ 0.0230	0.0	+ 0.0100	+ 0.0144	− 0.17	0.0	+ 0.24	+ 0.08	+ 0.15
1.0	± 0.4	+ 0.0610	+ 0.0320	+ 0.0480	+ 0.0524	− 0.46	+ 0.08	+ 1.13	+ 0.29	+ 1.04
1.0	± 0.3	+ 0.1070	+ 0.1670	+ 0.1480	+ 0.1407	− 0.80	+ 0.42	+ 3.48	+ 0.77	+ 3.87
1.0	± 0.2	+ 0.1310	+ 0.2600	+ 0.2150	+ 0.1961	− 0.98	+ 0.65	+ 5.05	+ 1.08	+ 5.80
1.0	± 0.1	+ 0.1425	+ 0.3200	+ 0.2510	+ 0.2263	− 1.07	+ 0.80	+ 5.90	+ 1.24	+ 6.87
1.0	0	+ 0.1460	+ 0.3400	+ 0.2640	+ 0.2356	− 1.10	+ 0.85	+ 6.20	+ 1.30	+ 7.25

SHEAR ALONG FIXED EDGES x = a AND y = ± 0.5b IN WALL SLAB

The magnitude and location of the maximum shear may readily be obtained from the above table.

HEEL SLAB



Determine the total vertical load W.

			Weight	x	lever arm	=	Moment
Wall Stem	= 1 x 18 x 150	=	2,700	x	9.5	=	25,650
	= 0.5 x 18 x 150	=	1,350	x	8.67	=	11,705
Footing	= 2 x 14 x 150	=	4,200	x	7	=	29,400
Moist Earth	= 8 x 20 x 125	=	20,000	x	4	=	80,000
	= 0.5 x 18 x 125	=	1,125	x	8.33	=	9,370
	= 1 x 2 x 125	=	250	x	8.5	=	2,125
Saturated Earth	= 8 x 13.5 x 15	=	1,620	x	4	=	6,480
	= 0.38 x 13.5 x 15	=	75	x	8.25	=	620
Counterfort	$= \frac{1 \times 4.5 \times 25}{14}$	=	10	x	8.17	=	80
	$= \frac{5 \times 13.5 \times 10}{14}$	=	50	x	5.50	=	275
Water	= 4 x 5.33 x 62.4	=	1,330	x	12	=	15,960
		ΣW =	32,710			ΣM =	181,665

$$x = \frac{\Sigma M}{\Sigma W} = \frac{181,665}{32,710} \quad 5.55 \text{ ft}$$

Determine the horizontal load H.

			Weight	x	lever arm	=	Moment
Water	$= -\frac{7.33^2 \times 62.4}{2}$	=	− 1,675	x	2.44	=	− 4,090
Surcharge	= 130 x 20	=	2,600	x	10	=	26,000
Earth Load	$= \frac{20^2 \times 65}{2}$	=	13,000	x	6.67	=	86,710
Pore Pressure Load	$= \frac{15.5^2 \times 20}{2}$	=	2,405	x	5.17	=	12,435
		ΣW =	16,330			ΣM =	121,055

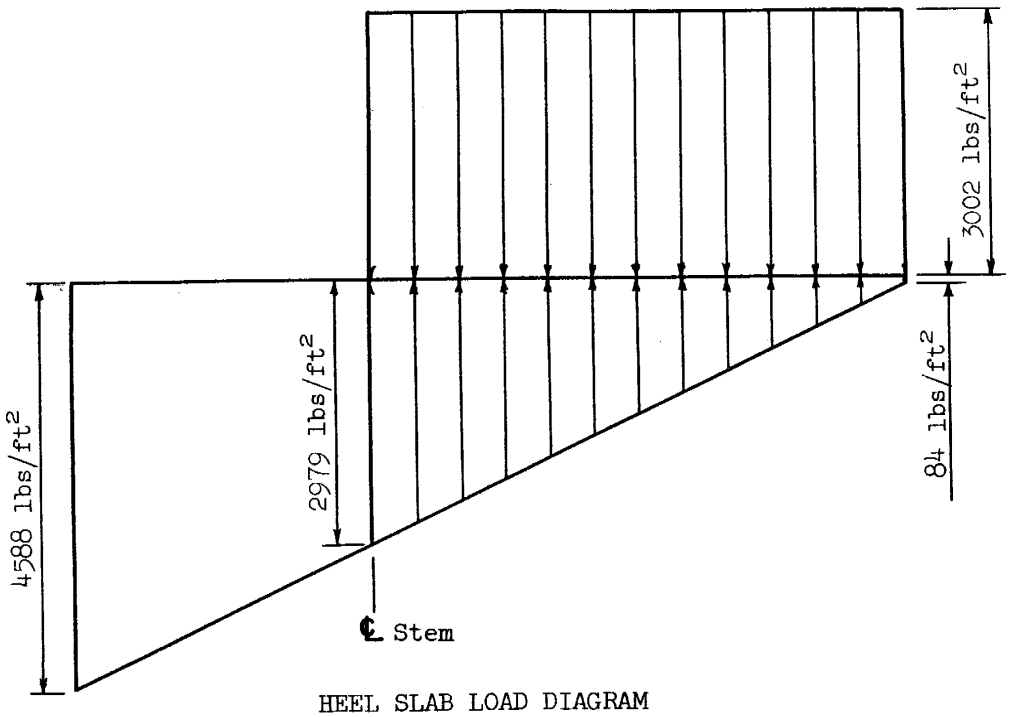
$$x' = \frac{(32,710)(8.45) - (16,330)(7.41)}{32,710} = 4.75 \text{ ft}$$

$$e = 7.00 - 4.75 = 2.25 \text{ ft}$$

STRUCTURAL DESIGN : DESIGN EXAMPLE ;
Heel slab dimensions and component loads

$$\text{Max Pressure} = \left(\frac{W}{b}\right)\left(1 + \frac{6e}{b}\right) = \left(\frac{32,710}{14}\right)\left(1 + \frac{(6)(2.25)}{14}\right) = 4588 \text{ lbs/ft}^2$$

$$\text{Min Pressure} = \left(\frac{W}{b}\right)\left(1 - \frac{6e}{b}\right) = \left(\frac{32,710}{14}\right)\left(1 - \frac{(6)(2.25)}{14}\right) = 84 \text{ lbs/ft}^2$$



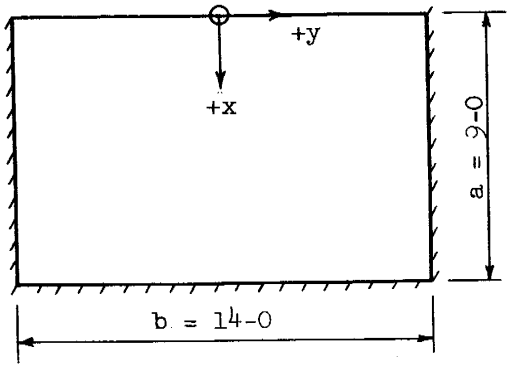
HEEL SLAB LOAD DIAGRAM

$$p_v = (4588 - 84)\left(\frac{9}{14}\right) = 2895 \text{ lbs/ft}^2$$

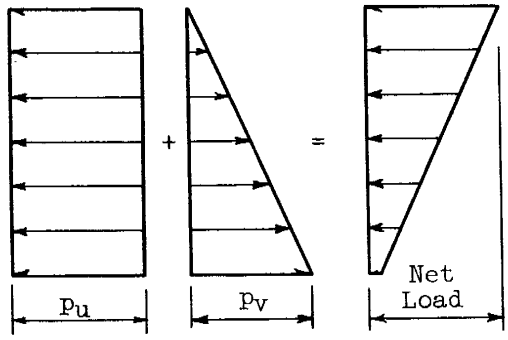
$$p_u = (20)(125) + (13.5)(15) + (2)(150) - 84 = 2918 \text{ lbs/ft}^2$$

$$p_v a^2 = (2.895)(9)^2 = 234.5 \text{ ft kips/ft}$$

$$p_u a^2 = (2.918)(9)^2 = 236.4 \text{ ft kips/ft}$$



IDEALIZED HEEL SLAB DIMENSIONS



COMPONENT HEEL SLAB LOADS

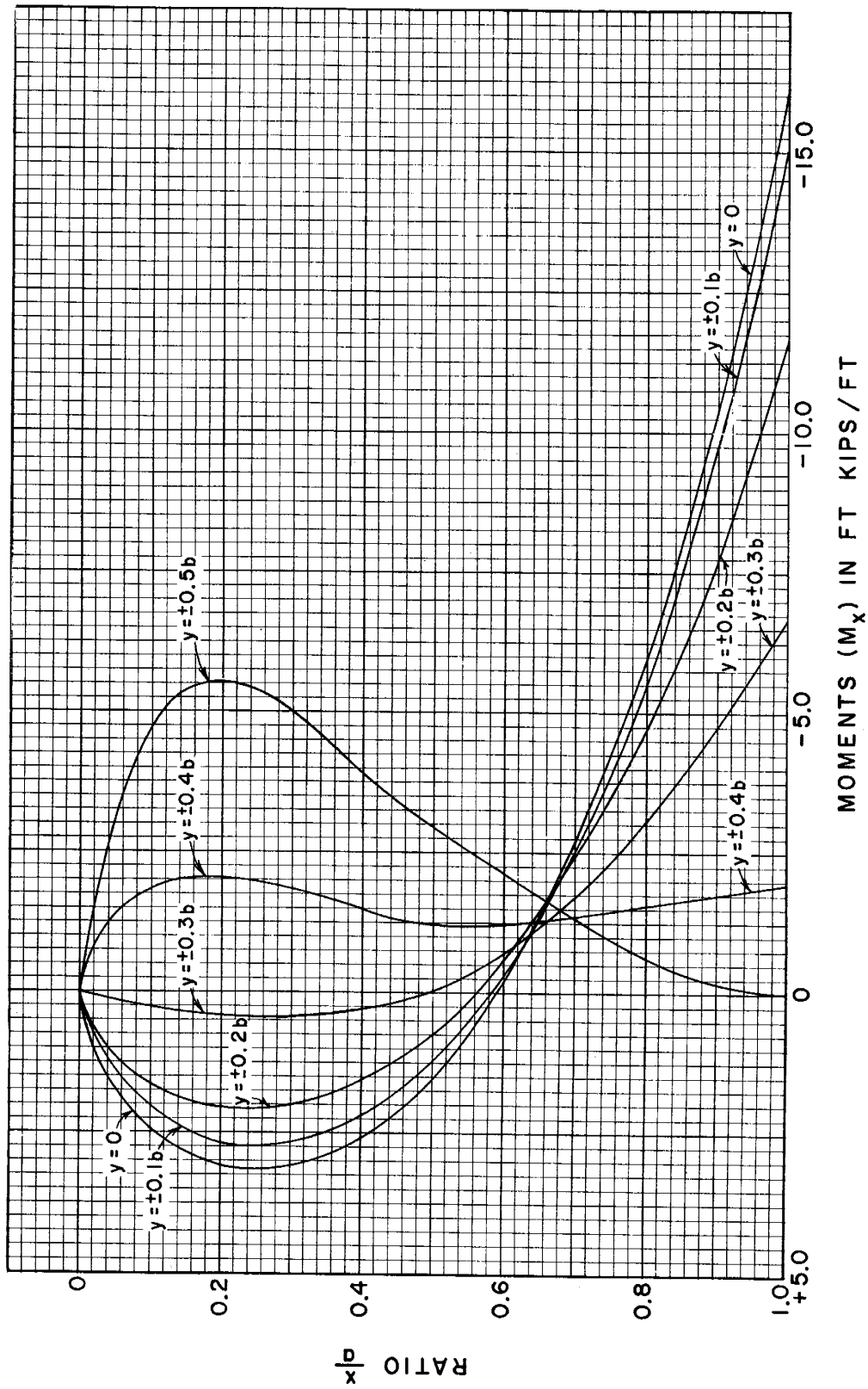
STRUCTURAL DESIGN : DESIGN EXAMPLE ; Moments (M_x) in heel slab

Values	$pa^2 \rightarrow$	Moment Coefficient		Moment (ft kips)		Total Moment ft kips
		236.4	− 234.5			
$\pm \frac{y}{b}$	$\frac{x}{a}$	p_u	p_v	M_u	M_v	
0	0	0	0	0	0	0
0	0.2	+ 0.0200	+ 0.0070	+ 4.73	− 1.64	+ 3.09
0	0.4	+ 0.0250	+ 0.0142	+ 5.91	− 3.33	+ 2.58
0	0.6	+ 0.0130	+ 0.0142	+ 3.07	− 3.33	− 0.26
0	0.8	− 0.0300	− 0.0048	− 7.09	+ 1.13	− 5.96
0	1.0	− 0.1290	− 0.0610	−30.50	+14.30	−16.20
± 0.1	0	0	0	0	0	0
± 0.1	0.2	+ 0.0180	+ 0.0067	+ 4.26	− 1.57	+ 2.69
± 0.1	0.4	+ 0.0230	+ 0.0138	+ 5.44	− 3.24	+ 2.20
± 0.1	0.6	+ 0.0120	+ 0.0133	+ 2.84	− 3.12	− 0.28
± 0.1	0.8	− 0.0275	− 0.0041	− 6.50	+ 0.96	− 5.54
± 0.1	1.0	− 0.1210	− 0.0575	−28.60	+13.48	−15.12
± 0.2	0	0	0	0	0	0
± 0.2	0.2	+ 0.0130	+ 0.0043	+ 3.07	− 1.01	+ 2.06
± 0.2	0.4	+ 0.0170	+ 0.0105	+ 4.02	− 2.46	+ 1.56
± 0.2	0.6	+ 0.0090	+ 0.0117	+ 2.13	− 2.74	− 0.61
± 0.2	0.8	− 0.0220	− 0.0020	− 5.20	+ 0.47	− 4.73
± 0.2	1.0	− 0.0970	− 0.0482	−22.93	+11.30	−11.63
± 0.3	0	0	0	0	0	0
± 0.3	0.2	+ 0.0030	+ 0.0013	+ 0.71	− 0.30	+ 0.41
± 0.3	0.4	+ 0.0070	+ 0.0059	+ 1.65	− 1.38	+ 0.27
± 0.3	0.6	+ 0.0048	+ 0.0078	+ 1.13	− 1.83	− 0.70
± 0.3	0.8	− 0.0133	− 0.0005	− 3.14	+ 0.12	− 3.02
± 0.3	1.0	− 0.0615	− 0.0335	−14.54	+ 7.86	− 6.68
± 0.4	0	0	0	0	0	0
± 0.4	0.2	− 0.0117	− 0.0031	− 2.77	+ 0.73	− 2.04
± 0.4	0.4	− 0.0072	− 0.0009	− 1.70	+ 0.21	− 1.49
± 0.4	0.6	− 0.0038	+ 0.0012	− 0.90	− 0.28	− 1.18
± 0.4	0.8	− 0.0068	− 0.0002	− 1.61	+ 0.05	− 1.56
± 0.4	1.0	− 0.0225	− 0.0145	− 5.32	+ 3.40	− 1.92
± 0.5	0	0	0	0	0	0
± 0.5	0.2	− 0.0323	− 0.0091	− 7.64	+ 2.13	− 5.51
± 0.5	0.4	− 0.0249	− 0.0088	− 5.89	+ 2.06	− 3.83
± 0.5	0.6	− 0.0160	− 0.0071	− 3.78	+ 1.66	− 2.12
± 0.5	0.8	− 0.0058	− 0.0033	− 1.37	+ 0.77	− 0.60
± 0.5	1.0	0	0	0	0	0

STRUCTURAL DESIGN : DESIGN EXAMPLE ; Moments (M_y) in heel slab

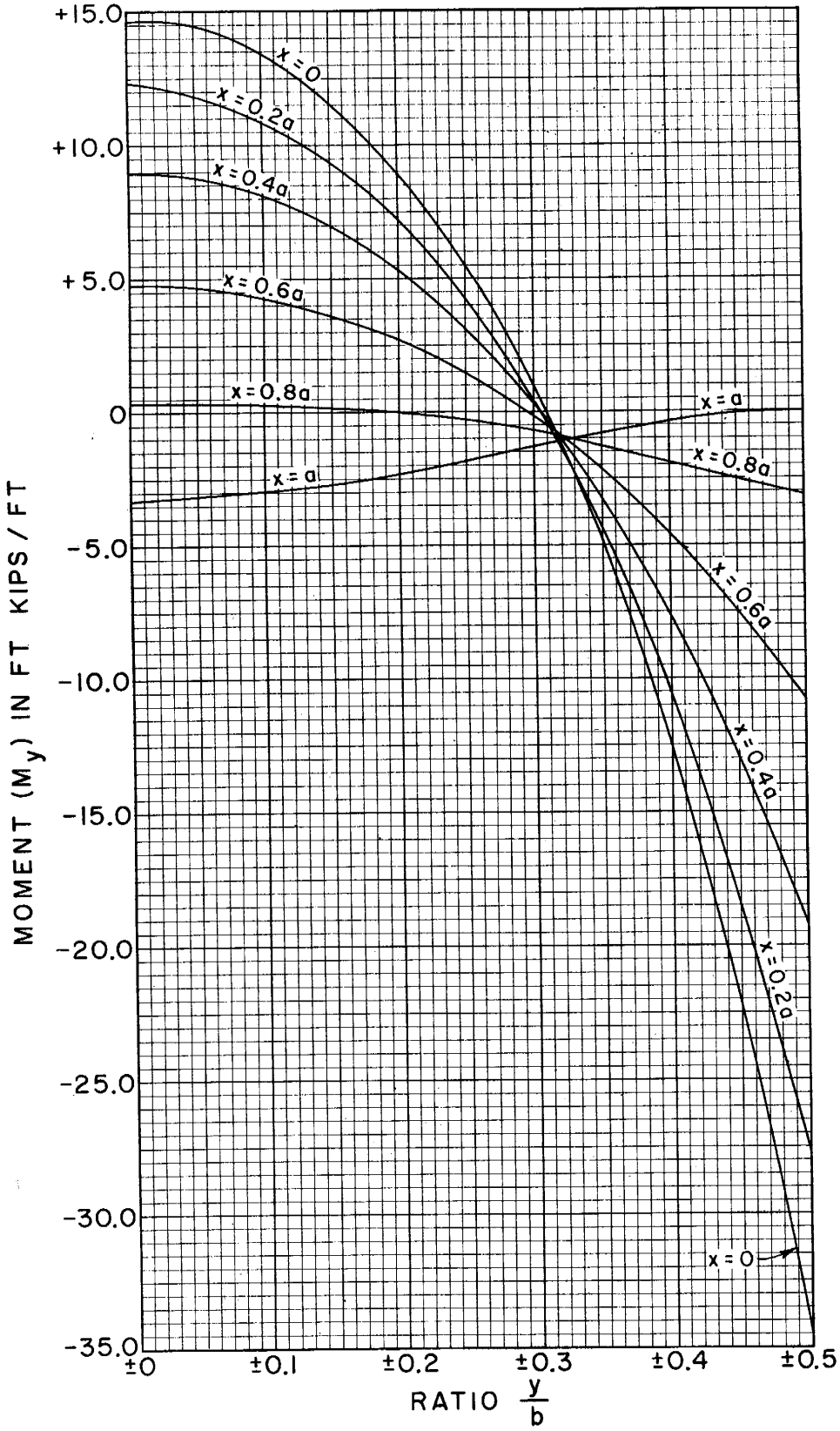
Values		Moment Coefficient		Moment (ft kips)		Total Moment ft kips
	$pa^2 \rightarrow$	236.4	− 234.5			
$\frac{x}{a}$	$\pm \frac{y}{b}$	p_u	p_v	M_u	M_v	
0	0	+ 0.0840	+ 0.0225	+ 19.86	− 5.28	+ 14.58
0	± 0.1	+ 0.0760	+ 0.0195	+ 17.97	− 4.57	+ 13.40
0	± 0.2	+ 0.0500	+ 0.0125	+ 11.82	− 2.93	+ 8.89
0	± 0.3	+ 0.0030	− 0.0013	+ 0.71	+ 0.30	+ 1.01
0	± 0.4	− 0.0740	− 0.0206	− 17.49	+ 4.83	− 12.66
0	± 0.5	− 0.1880	− 0.0430	− 44.44	+ 10.08	− 34.36
0.2	0	+ 0.0730	+ 0.0212	+ 17.26	− 4.97	+ 12.29
0.2	± 0.1	+ 0.0650	+ 0.0192	+ 15.37	− 4.50	+ 10.87
0.2	± 0.2	+ 0.0430	+ 0.0126	+ 10.17	− 2.95	+ 7.22
0.2	± 0.3	+ 0.0020	+ 0.0007	+ 0.47	− 0.16	+ 0.31
0.2	± 0.4	− 0.0630	− 0.0182	− 14.89	+ 4.27	− 10.62
0.2	± 0.5	− 0.1620	− 0.0452	− 38.30	+ 10.60	− 27.70
0.4	0	+ 0.0570	+ 0.0192	+ 13.47	− 4.50	+ 8.97
0.4	± 0.1	+ 0.0520	+ 0.0178	+ 12.29	− 4.17	+ 8.12
0.4	± 0.2	+ 0.0350	+ 0.0127	+ 8.27	− 2.98	+ 5.29
0.4	± 0.3	+ 0.0040	+ 0.0028	+ 0.94	− 0.66	+ 0.28
0.4	± 0.4	− 0.0470	− 0.0147	− 11.11	+ 3.45	− 7.66
0.4	± 0.5	− 0.1250	− 0.0439	− 29.55	+ 10.29	− 19.26
0.6	0	+ 0.0335	+ 0.0137	+ 7.92	− 3.21	+ 4.71
0.6	± 0.1	+ 0.0308	+ 0.0130	+ 7.28	− 3.05	+ 4.23
0.6	± 0.2	+ 0.0215	+ 0.0104	+ 5.08	− 2.44	+ 2.64
0.6	± 0.3	+ 0.0035	+ 0.0040	+ 0.83	− 0.94	− 0.11
0.6	± 0.4	− 0.0285	− 0.0090	− 6.73	+ 2.11	− 4.62
0.6	± 0.5	− 0.0805	− 0.0352	− 19.03	+ 8.25	− 10.78
0.8	0	+ 0.0045	+ 0.0030	+ 1.06	− 0.70	+ 0.36
0.8	± 0.1	+ 0.0041	+ 0.0030	+ 0.97	− 0.70	+ 0.27
0.8	± 0.2	+ 0.0028	+ 0.0028	+ 0.66	− 0.66	0
0.8	± 0.3	− 0.0012	+ 0.0016	− 0.28	− 0.38	− 0.66
0.8	± 0.4	− 0.0112	− 0.0030	− 2.65	+ 0.70	− 1.95
0.8	± 0.5	− 0.0292	− 0.0163	− 6.90	+ 3.82	− 3.08
1.0	0	− 0.0260	− 0.0122	− 6.15	+ 2.86	− 3.29
1.0	± 0.1	− 0.0242	− 0.0116	− 5.72	+ 2.72	− 3.00
1.0	± 0.2	− 0.0193	− 0.0097	− 4.56	+ 2.27	− 2.29
1.0	± 0.3	− 0.0123	− 0.0067	− 2.91	+ 1.57	− 1.34
1.0	± 0.4	− 0.0045	− 0.0030	− 1.06	+ 0.70	− 0.36
1.0	± 0.5	0	0	0	0	0

STRUCTURAL DESIGN : DESIGN EXAMPLE ;
Moments (M_x) in heel slab



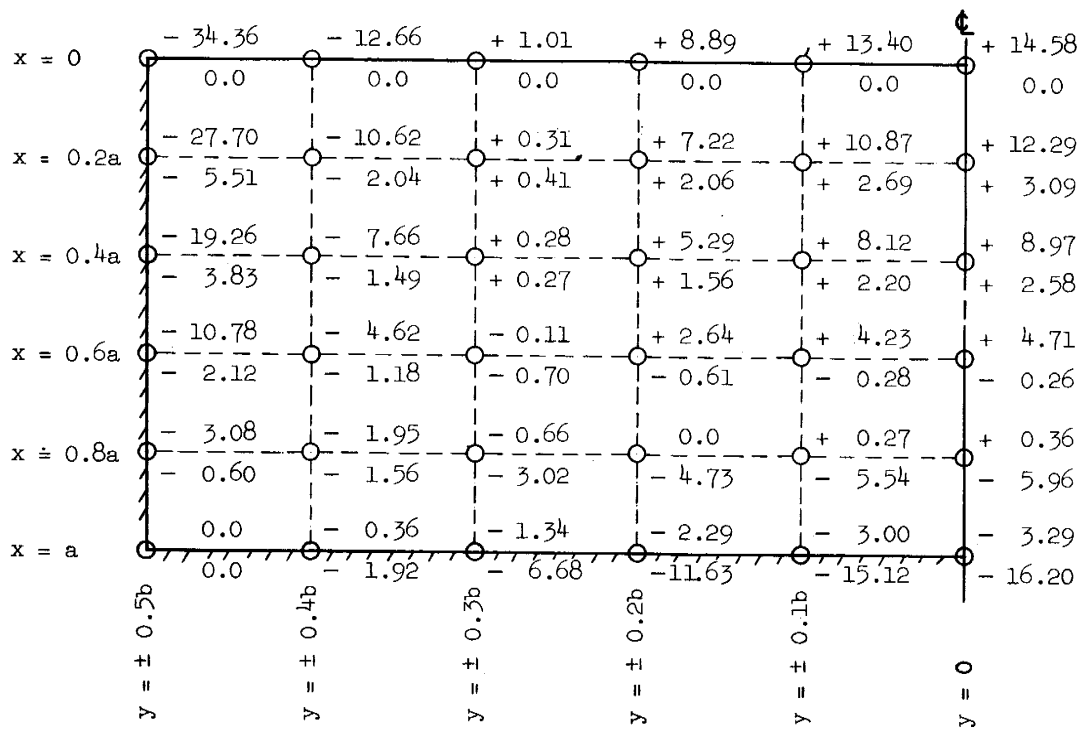
REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - DESIGN SECTION	STANDARD DWG. NO. ES- 104 SHEET 83 OF 85 DATE 4-13-56

STRUCTURAL DESIGN : DESIGN EXAMPLE ; Moments (M_y) in heel slab

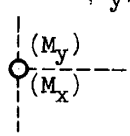


REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - DESIGN SECTION	STANDARD DWG. NO. ES-104 SHEET 84 OF 85 DATE 4-13-56

STRUCTURAL DESIGN ; DESIGN EXAMPLE ;
Moments and shears in heel slab



MOMENTS (M_y) AND (M_x) IN ft kips/ft IN HEEL SLAB



Values pa →		Shear Coefficient		Shear kips/ft		Total Shear kips/ft
		26.3	- 26.1			
$\frac{x}{a}$	$\pm \frac{y}{b}$	p_u	p_v	s_u	s_v	
0	± 0.5	+ 0.90	+ 0.114	+ 23.7	- 3.0	+ 20.7
0.2	± 0.5	+ 0.805	+ 0.213	+ 21.2	- 5.6	+ 15.6
0.4	± 0.5	+ 0.603	+ 0.243	+ 15.9	- 6.3	+ 9.6
0.6	± 0.5	+ 0.435	+ 0.252	+ 11.4	- 6.6	+ 4.8
0.8	± 0.5	+ 0.110	+ 0.130	+ 2.9	- 3.4	- 0.5
1.0	± 0.5	- 0.075	- 0.020	- 2.0	+ 0.5	- 1.5
1.0	± 0.4	+ 0.085	+ 0.130	+ 2.2	- 3.4	- 1.2
1.0	± 0.3	+ 0.346	+ 0.271	+ 9.1	- 7.1	+ 2.0
1.0	± 0.2	+ 0.542	+ 0.355	+ 14.3	- 9.3	+ 5.0
1.0	± 0.1	+ 0.655	+ 0.399	+ 17.2	- 10.4	+ 6.8
1.0	0.0	+ 0.692	+ 0.411	+ 18.2	- 10.7	+ 7.5

SHEAR ALONG FIXED EDGES x = a AND y = ± 0.5b IN HEEL SLAB

The magnitude and location of the maximum shear may be readily obtained from the above table.

4. Reinforced Concrete

4.1 Classes of Reinforced Concrete. The class of concrete to be used in any specific job should be based on a study of the job requirements as to strength and durability. Many factors affect the quality of concrete; the best materials and design do not produce excellent concrete without high quality methods of construction.

Nine classes of concrete are presently established. They cover the various conditions of design and construction encountered by the Soil Conservation Service. For Class 5000, Class 4000, Class 3000, and Class 2500 concrete the Contractor is responsible for the design of the concrete mix. For Class 5000X, Class 4000X, Class 3000X, Class 3000M, and Class 2500X concrete the Engineer is responsible for the design of the concrete mix. The following is a general guide to these concrete classes and their use.

Class 5000 or 5000X concrete -- for special structures, for precast or prestressed construction, for extreme exposure conditions.

Class 4000 or 4000X concrete -- for standard types and sizes of structures, for moderate exposure conditions.

Class 3000 or 3000X concrete -- for small simple structures, for mass foundations.

Class 3000M -- for minor concrete structures in which the quantity of concrete is less than 5 yards and where the location of the concrete will permit easy maintenance or replacement.

Class 2500 or 2500X concrete -- for small structures built by the farmer or unskilled labor, for plain concrete construction.

Guide Construction Specifications 31. Concrete, 32. Concrete for Minor Structures, and 34. Steel Reinforcement (NEH Section 20) state the technical and workmanship requirements for the operations required in reinforced concrete construction. These specifications include such items as:

Air Content and Consistency
Design of Concrete Mix
Inspection and Testing
Mixing, Conveying, Placing, Consolidating and Curing Concrete
Preparation and Removal of Forms
Fabrication and Placing of Reinforcement.

Guide Material Specifications 531. Portland Cement, 522. Aggregate for Portland Cement Concrete, and 539. Steel Reinforcement (NEH Section 20) state the quality of materials to be incorporated in the construction.

4.2 Design Codes and Criteria:

4.2.1 General Code to be Used. The American Concrete Institute Standard "Building Code Requirements for Reinforced Concrete" (ACI 318-77), Appendix B - Alternate Design Method is used as the design code for working stress design except as modified in 4.2.2.

4.2.2 Other Design Criteria

(a) The allowable extreme fiber unit stress in compression in flexural members is: $f_c = 0.40\ f'_c$.

(b) The allowable tensile unit stress in reinforcement is: $f_s = 20000$ psi. The design yield strength is: $f_y = 40000$ psi.

(c) Members subjected to bending and direct compressive force, in which the eccentricity ($e = M/N$) is not less than that causing balanced working stresses, are designed on the basis of recognized theory of cracked sections. The tensile steel may be stressed to its allowable value; the concrete stress may not exceed its allowable value.

(d) In doubly reinforced flexural members, the modular ratio, E_s/E_c is used to transform compression reinforcement for stress computations.

(e) The minimum clear concrete cover over reinforcement is two inches, except when concrete is deposited on or against earth, the minimum clear concrete cover is three inches. However, in structural design of slabs or beams without web reinforcement, the distance from the surface of the concrete to the centerline of the nearest reinforcing steel may be taken as 2-1/2 or 3-1/2 inches, as the case may be, to simplify the determination of the effective depth, for all bars one inch or less in diameter.

Consideration should be given to increasing the cover when a concrete surface is exposed to high velocities and the water carries abrasive materials.

(f) Reinforcing steel is required in both faces and in both (orthogonal) directions in all concrete slabs and walls, except that only one grid of reinforcing is required in concrete linings of trapezoidal channels. This steel serves either as principal reinforcement or as temperature and shrinkage reinforcement. The minimum steel areas for slabs and walls having thickness equal to or less than 32 inches, in each face and in each direction, expressed as the ratio, ρ_t, of reinforcement area, A_s, to gross concrete area, bt, are as follows:

1. The steel in the direction in which the distance between expansion or contraction joints does not exceed thirty feet,

   $\rho_t = 0.002$ in an exposed face

   $\rho_t = 0.001$ in an unexposed face.

2. The steel in the direction in which the distance between expansion or contraction joints exceeds thirty feet,

$$\rho_t = 0.003 \text{ in an exposed face}$$

$$\rho_t = 0.002 \text{ in an unexposed face.}$$

The minimum steel areas for slabs and walls having thicknesses greater than 32 inches are computed as though the thickness were 32 inches.

When expansion or contraction in a member is restrained along any line, the concept of equivalent distance between expansion or contraction joints should be used to determine the required steel ratio, ρ_t. The equivalent distance is taken as double the perpendicular distance from the line of restraint to the far edge or line of support of the member.

When the surface of a wall or slab will be exposed for a considerable period during construction, the steel provided should satisfy requirements for an exposed face.

(g) Where a single grid of reinforcement is used, as permitted above, the steel ratio, ρ_t, is the sum of that listed for both faces.

(h) Splices and development lengths for temperature and shrinkage reinforcement are designed for the design yield strength, f_y.

(i) The maximum spacing of principal steel is twice the thickness of the wall or slab, but not more than 18 inches. The maximum spacing of temperature steel is three times the thickness of the wall or slab, but not more than 18 inches.

(j) Where principal steel is required in only one direction, it is ordinarily placed nearer the concrete surface than the temperature steel. Where principal steel is required in both directions, the steel which carries the larger moment is, ordinarily placed nearer the concrete surface. Where principal steel is required in neither direction, the temperature steel parallel to the longer dimension of the slab or wall is ordinarily placed nearer the concrete surface.

(k) The clear distance between parallel bars in a layer is not less than the bar diameter, 1-1/3 times the maximum size of the coarse aggregate, nor 1 inch. Where parallel reinforcement is placed in two or more layers, bars in the upper layers are placed directly above bars in the bottom layer. The clear distance between layers is not less than 1 inch. The clear distance between bars also applies to the clear distance between a contact lap splice and adjacent splices or bars.

(ℓ) The calculated tension or compression in any bar at any section must be developed on each side of that section by proper embedment length, end anchorage, or hooks. Hooks may be used in developing bars in tension. Hooks are not effective in developing bars in compression.

(m) The development lengths, for design yield strength f_y = 40 ksi, for bars #11 and smaller, are given as follows.

For tension top bars, the development length, ℓ_d, is the larger of $1.4(1600\ A_b/\ \sqrt{f'_c})$ or $1.4(16\ d_b)$ but not less than 12 inches.

For all other tension bars, the development length, ℓ_d, is the larger of $(1600\ A_b/\ \sqrt{f'_c})$ or $(16\ d_b)$ but not less than 12 inches.

Top bars are defined as horizontal bars so placed that more than 12 inches of concrete is cast in the member below the bars. Tension bars spaced laterally not less than 6 inches on centers, and bars with at least 3 inches clear from face of member to first bar, may use 0.8 the development length given above but not less than 12 inches.

For compression bars, the development length, ℓ_d, is the larger of $(800\ d_b/\ \sqrt{f'_c})$ or $(12\ d_b)$ but not less than 8 inches.

In the above relations: A_b is the area of an individual bar in square inches, and d_b is the nominal diameter of a bar in inches.

(n) Splices should be made at or close to points of inflection if it is practical to do so. Lap splices shall not be used for bars larger than #11. Bars in a noncontact splice shall not be farther apart than 1/5 the required length of lap nor 6 inches. Lap splices are designed for the design yield strength f_y = 40 ksi.

(o) Three classes of tension lap splices are established. The minimum length of lap is determined as a multiplier for the class times the development length, ℓ_d, but not less than 30 bar diameters. The classes and minimum lengths are:

Class A splice . . . . . 1.0 ℓ_d

Class B splice . . . . . 1.3 ℓ_d

Class C splice . . . . . 1.7 ℓ_d.

The splice class required depends upon the stress level in the reinforcement to be spliced and the portion of the total reinforcement to be spliced at the cross section.

If the area of tensile steel provided at the splice location is equal to or more than twice that required by analysis (low tensile stress in the reinforcement) and not more than 75 percent of the bars are to be lap spliced within the required lap splice length, a Class A splice may be used. If more than 75 percent of the bars are to be lap spliced within the required lap splice length, a Class B splice is required.

If the area of tensile steel provided at the splice location is less than twice that required by analysis (high tensile stress in reinforcement) and not more than 50 percent of the bars are to be lap spliced within the required lap splice length, a Class B splice may be used. If more than 50 percent of the bars are to be lap spliced within the required lap splice length, a Class C splice is required.

(p) The minimum length for compressive lap splices is the larger of 24 bar diameters or 12 inches. For $f'_c < 3000$ psi, the lap length is increased by 1/3.

(q) Critical sections for development of reinforcement are at points of maximum stress and at points where adjacent reinforcement terminates.

Except at supports of simple spans and at the free end of cantilevers, every reinforcing bar is extended beyond the point at which it is no longer needed to resist flexural stress, for a distance equal to the effective depth of the member or 12 bar diameters, whichever is greater.

Continuing reinforcement has an embedment length not less than the development length, ℓ_d, beyond the point where bent or terminated tension reinforcement is no longer needed to resist flexural stress.

At least 1/3 the positive moment reinforcement in simple spans and 1/4 the positive moment reinforcement in continuous spans extends along the same face of the span into the support at least 6 inches.

At least 1/3 the negative moment reinforcement at a support extends beyond the extreme position of the point of inflection a distance not less than the effective depth of the member, 12 bar diameters, or 1/16 the clear span, whichever is greater.

(r) Sufficient longitudinal tension steel perimeter is provided at every section so that flexural bond stresses do not exceed allowable values. Critical sections for flexural bond stresses occur where the rate of change of moment is greatest or where the steel perimeter is least, or both. For simple spans, critical sections are at the faces of supports. For continuous spans: for negative steel, critical sections are located at faces of supports and at locations where bars terminate; for positive steel, critical sections are at points of inflection.

(s) To aid in the control of flexural cracking in beams and one-way slabs, cross sections at both maximum positive and maximum negative moment locations are proportioned so that the quantity, Z, given by

$$Z = f_s \sqrt[3]{d_c A}$$

does not exceed 130. In the relation: f_s is the calculated stress in the reinforcement in ksi (in lieu of calculations, the value of f_s may be taken as the allowable stress), d_c is the thickness of concrete cover in inches measured from the extreme tension fiber to the center of the longitudinal bar located closest to the extreme fiber, and A is the effective tension area of concrete per bar in square inches. A is determined as the tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as the reinforcement, divided by the number of bars.

4.2.3 State and other Local Codes. State and local codes may not be satisfied by the above mentioned code or design criteria. The local design engineer should be familiar with the state and local codes and it should be his responsibility to see that these codes are met.

4.3 Design Procedures

4.3.1 Reference Materials. Procedures for design may be found in several manuals and textbooks, among which are:

"Reinforced Concrete Design" by Sutherland and Reese

"Reinforced Concrete Structures" by Peabody

"Reinforced Concrete Fundamentals" by Ferguson

"Theory and Practice of Reinforced Concrete" by Dunham

"Design of Concrete Structures" by Winter, et al

"Reinforced Concrete Design" by Wang and Salmon

Various design handbooks by ACI

Various publications of the Portland Cement Association

"CRSI handbook" by Concrete Reinforcing Steel Institute.

Numerous computation aids have been developed to simplify and speed up design work by eliminating the need of solving various design formulas. These aids follow, and their use is illustrated by 4.3.2.

4.3.2 Example Problems. Example problems for:

(a) Simple Bending
(b) Bending and Direct Compressive Force
(c) Bending and Direct Tensile Force
(d) Beam Shear (as a Measure of Diagonal Tension)
(e) Flexural Bond
(f) Temperature and Shrinkage Steel

are solved below. All problems use $f'_c = 4000$ psi, $n = 8.0$, $f_s = 20$ ksi, and concrete cover from center of steel = 2.5 in.

(a) Simple Bending

1. Problem: Find the effective depth and steel area which produces balanced stresses at a slab section where the moment is 14.0 ft kips per ft of width.

   Solution: On ES-164, sheet 1 at moment $M_s = M = 14.0$ ft kips and the balanced stress line, read $d = 7.20$ in. and $A = A_s = 1.34$ sq in. A practical solution is $d = 7.5$ in., $t = 7.5 + 2.5 = 10.0$ in. and #8 bars at 7 in. on centers.

2. Problem: Determine the allowable moment for a slab 8 in. thick, reinforced with #4 @ 12 in. on centers.

   Solution: $d = 8 - 2.5 = 5.5$ in.; from ES-46 $A_s = 0.20$ sq in per ft

   On ES-164, sheet 1 at effective depth $d = 5.5$ in. and $A = A_s = 0.20$ sq in., read $M = M_s = 1.70$ ft kips per ft of width.

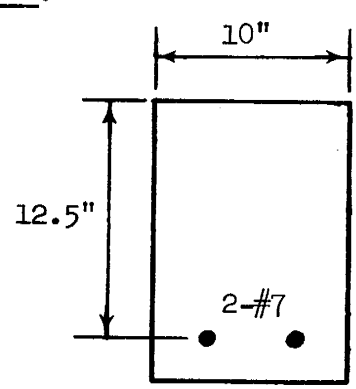
3. Problem: Find the steel area required for a beam having a width of 10 in., an effective depth of 12.5 in., and a moment of 22 ft kips.

   Solution: Moment per ft of width = 22(12/10) = 26.4 ft kips

   On ES-164, sheet 1 at moment $M_s = M = 26.4$ ft kips and $d = 12.5$ in., read $A = A_s = 1.42$ sq in. Steel area required for given beam = 1.42(10/12) = 1.18 sq in.
   Select 2 – #7 bars, $A_s = 1.20$ sq in.

4. Problem: Check the solution of problem (a)3 by using the transformed section and the common flexure formula.

   Solution:



$M = 22$ ft kips

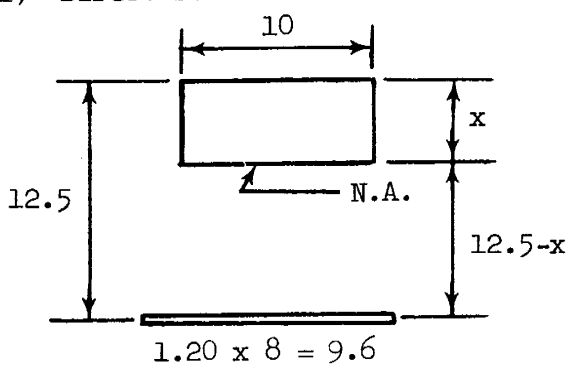
$A_s = 1.20$ sq in

Determine the location of the neutral axis. For simple bending, the neutral axis is the same as the center of gravity axis. This may be located in several ways, but two are shown.

(1) Trial and error solution for the location of the neutral axis. Assume the neutral axis is 3.8 in. below the top of the beam, take moments about the top of the beam to obtain a new estimate of the distance to the neutral axis.

		Area	Arm	Moment
Compression	10.00 x 3.8	= 38.0	1.9	72.2
Transformed Tension	1.20 x 8	= 9.6	12.5	120.0
		47.6		192.2/47.6 = 4.04
Corrective Area	10.00 x 0.24	= 2.4	3.92	9.4
		50.0		201.6/50.0 = 4.03"

(2) Direct solution for location of neutral axis. Balance the moment of the compressive area and the moment of the transformed steel area about the unknown location of the neutral axis.



$$(10x)\left(\frac{x}{2}\right) = 9.6(12.5 - x)$$

$$5x^2 + 9.6x = 120$$

$$x = 4.03"$$

Compute the Moment of Inertia about the neutral axis.

$$\frac{1}{3}(10)(4.03)^3 = 218$$

$$9.6(8.47)^2 = 688$$

$$I = 906 \text{ in}^4$$

Compute the unit stresses.

$$f_c = \frac{Mx}{I} = \frac{(22 \times 12 \times 1000)(4.03)}{906} = 1170 \text{ psi} < 1600 \text{ psi OK}$$

$$f_s = \frac{nM(d - x)}{I} = \frac{8(22 \times 12)(8.47)}{906} = 19.7 \text{ ksi} < 20 \text{ ksi OK}$$

5. Problem: Find the steel areas required at a slab section having an effective depth of 15 in. and a moment of 65 ft kips per ft of width.

Solution: On ES-164, sheet 1 at moment $M_s = M = 65.0$ ft kips and $d = 15.0$ in., read required steel areas $A = A_s = 3.0$ sq in. and $A_s' = 0.70$ sq in.

From ES-46 select:

#9 @ 4" on centers for tensile steel.
#8 @ 12" on centers for compressive steel.

(b) Bending and Direct Compressive Force

1. Problem: Find the steel area required at a slab section having a total depth of 12 in., a moment of 15.5 ft kips per ft of width and a direct compressive force of 13.7 kips per ft of width.

Solution: $d = 12 - 2.5 = 9.5$ in.; $d'' = (12/2) - 2.5 = 3.5$ in.

$$M_s = M + \frac{Nd''}{12} = 15.5 + \frac{13.7 \times 3.5}{12} = 19.5 \text{ ft kips}$$

On ES-164, sheet 1 at moment $M_s = 19.5$ ft kips and $d = 9.5$ in., read $A = 1.40$ sq in.

$$A_s = A - \frac{N}{20} = 1.40 - \frac{13.7}{20} = 0.72 \text{ sq in.}$$

From ES-46 select:

#7 @ 10" on centers.

2. Problem: Same as problem (b)1 except moment is 22.0 ft kips per ft of width.

Solution: $M_s = M + \frac{Nd''}{12} = 22.0 + \frac{13.7 \times 3.5}{12} = 26.0$ ft kips

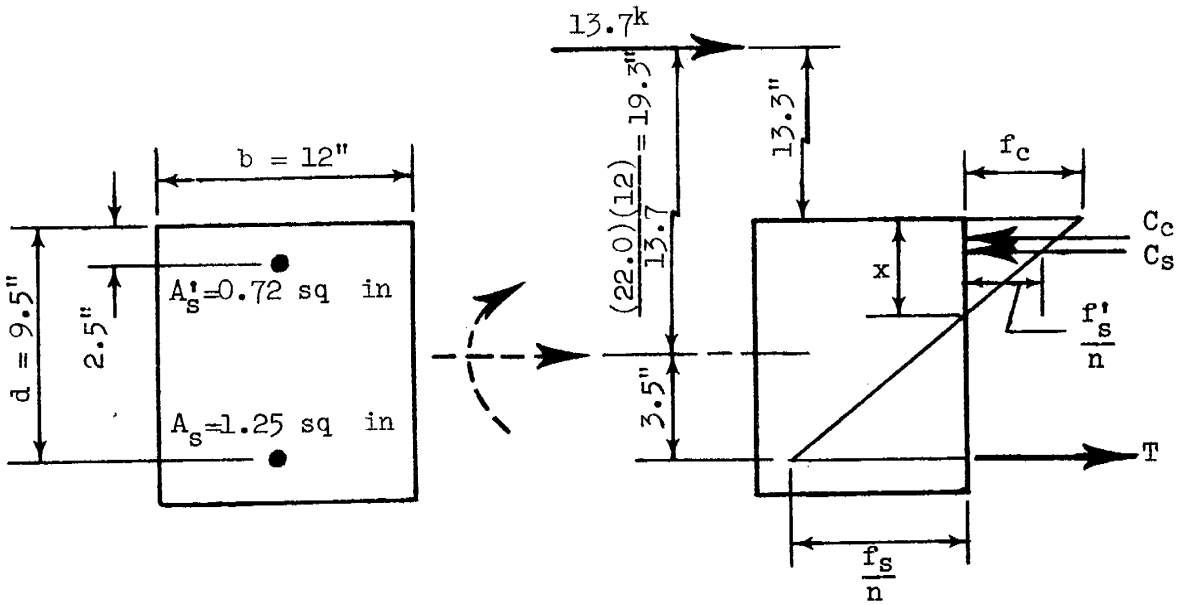
On ES-164, sheet 1 at moment $M_s = 26.0$ ft kips and $d = 9.5$ in., read required areas $A = 1.90$ sq in and

$A_s' = 0.70$ sq in. Thus $A_s = A - \frac{N}{20} = 1.90 - \frac{13.7}{20} = 1.22$ sq in.

From ES-46 select #7 @ 10" on centers staggered with #6 @ 10" on centers for tensile steel, $A_s = 0.72 + 0.53 = 1.25$ sq in. Select #7 @ 10" on centers for compressive steel, $A_s' = 0.72$ sq in.

3. Problem: Check the solution of problem (b)2 by computing the unit stresses.

Solution: Replace the moment, M = 22.0 ft kips and direct compressive force N = 13.7 kips, by an equivalent eccentric compressive force as shown.



The force relations in terms of concrete stress are:

$$C_c = \frac{1}{2} f_c bx = 6\ f_c x$$

$$C_s = \frac{f'_s}{n}(n-1)A'_s = f_c\left(\frac{x-2.5}{x}\right)(n-1)A'_s = 5.04 f_c\left(\frac{x-2.5}{x}\right)$$

$$T = \frac{f_s}{n}(n)A_s = f_c\left(\frac{9.5-x}{x}\right)nA_s = 10.0 f_c\left(\frac{9.5-x}{x}\right)$$

Moments about a point on the line of action of the eccentric force yield:

$$T(19.3 + 3.5) = C_c(13.3 + \tfrac{x}{3}) + C_s(13.3 + 2.5)$$

$$228\ f_c\left(\frac{9.5-x}{x}\right) = 6\ f_c x(13.3 + \tfrac{x}{3}) + 79.6\ f_c\left(\frac{x-2.5}{x}\right)$$

$$x^3 + 39.9x^2 + 153.8x = 1188$$

therefore, by trial, x = 3.75 in.

Summation of forces yields:

$$C_c + C_s - T = 13700$$

$$6\ f_c(3.75) + 5.04\ f_c\left(\frac{3.75 - 2.5}{3.75}\right) - 10\ f_c\left(\frac{9.5 - 3.75}{3.75}\right) = 13700$$

therefore, $f_c = 1550$ psi < 1600 psi OK

$$f_s = 8(1.550)\left(\frac{9.5 - 3.75}{3.75}\right)$$

$$= 19.0 \text{ ksi} < 20 \text{ ksi} \qquad \text{OK}$$

(c) Bending and Direct Tensile Force

1. Problem: Find the steel area required at a slab section having a total depth of 12 in., a moment of 22.0 ft kips per ft of width, and a direct tensile force of 13.7 kips per ft of width.

Solution:

$$d = 12 - 2.5 = 9.5 \text{ in.};\ d'' = (12/2) - 2.5 = 3.5 \text{ in.}$$

$$M_s = M - \frac{Nd''}{12} = 22.0 - \frac{13.7 \times 3.5}{12} = 18.0 \text{ ft kips}$$

On ES-164, sheet 1 at moment $M_s = 18.0$ ft kips and $d = 9.5$ in., read $A = 1.28$ sq in.

$$A_s = A + \frac{N}{20} = 1.28 + \frac{13.7}{20} = 1.97 \text{ sq in}$$

From ES-46 select #9 @ 6" on centers.

(d) Beam Shear (as a Measure of Diagonal Tension)

1. Problem: Determine the allowable total shear at the face of the support of a uniformly loaded beam having no web reinforcement if $b = 12$ in., $t = 13$ in., and the live plus dead loading is $q = 1.2$ kips per lineal ft.

Solution: $d = 13 - 2.5 = 10.5$ in.

On ES-164, sheet 2 at $d = 10.5$ in. and $q = 1.2$ kips per lineal ft, read $V_s = 9.88$ kips.

2. Problem: Same as problem (d)1 except b = 8 in.

Solution: Compute q for b = 12 in., $q = 1.2(\frac{12}{8}) = 1.8$ kips per lineal ft.

On ES-164, sheet 2 at d = 10.5 in. and q = 1.8 kips per lineal ft, read $V_s = 10.4$ kips. Compute V_s for b = 8 in.,

$V_s = 10.4(\frac{8}{12}) = 6.93$ kips.

3. Problem: Determine the depth required by shear, if no web reinforcement is used, when a uniformly loaded one-way slab having a clear span of 12.0 ft carries a live load of 2.0 kips per sq ft.

Solution: For the first trial neglect dead weight; then q = 2.0 kips per lineal ft and $V_s = \frac{1}{2}\,qL = \frac{1}{2} \times 2.0 \times 12 = 12.0$ kips. On ES-164, sheet 2 at $V_s = 12.0$ kips and q = 2.0 kips per lineal ft, read d = 11.9 in. Including the dead weight, d must be greater than 11.9 in.; therefore try d = 13.0 in., t = 13.0 + 2.5 = 15.5 in.,

and $q = 2.0 + 0.150(\frac{12 \times 15.5}{144}) = 2.19$ kips per lineal ft, so that

$V_s = \frac{1}{2} \times 2.19 \times 12 = 13.1$ kips. On ES-164, sheet 2 at $V_s = 13.1$ kips and q = 2.19 kips per lineal ft, read d = 12.8 in. Since $12.8 \leq 13.0$ use d = 13.0 in. and t = 15.5 in.

(e) Flexural Bond

"Top bars" are defined as horizontal bars so placed that more than 12 in. of concrete is cast in the member below the bars.

1. Problem: Determine the allowable total shear at the face of the support of a one-way simple slab having t = 13 in. and reinforced with #5 @ 12" on centers.

Solution: d = 13 − 2.5 = 10.5 in.; from ES-46 $\Sigma o = 1.96$ in. per ft of width. On ES-164, sheet 3, the chart for "Tension Bars Other Than Top Bars" at $\Sigma o = 1.96$ in. and the line for #5 bars, read V/d = 0.83 kips per in. Thus V = 0.83 x 10.5 = 8.72 kips per ft of width.

2. Problem: Determine the maximum spacing permitted by bond if #6 bars are used to reinforce a one-way cantilever slab having t = 15 in. and V = 11.9 kips per ft of width.

Solution: d = 15 − 2.5 = 12.5 in.; $V/d = \frac{11.9}{12.5} = 0.95$ kips per in.

On ES-164, sheet 3, the chart for "Tension Top Bars" V/d = 0.95 kips per in. and the line for #6 bars, read $\Sigma o = 3.80$ in.

The maximum allowable spacing is $s = 12(\frac{2.36}{3.80}) = 7.45$ in.

(f) Temperature and Shrinkage Steel

1. Problem: Determine the temperature and shrinkage steel required in the exposed face and the T and S steel required in the unexposed face of a 16 in. thick slab. The distance between joints in each face is less than 30 ft.

   Solution: For the exposed face, the minimum value of p_t is 0.002. Thus from ES-47 for t = 16 any of the following combinations of bar size and spacing might be selected: #4 @ 6", #5 @ 9", #6 @ 13", or #7 @ 18". For the unexposed face, the minimum value of p_t is 0.001. Thus from ES-47 any of the following might be selected: #3 @ 6", #4 @ 12", or #5 @ 18". Note that ES-164, sheet 1 can be used to determine the required areas of T and S steel; i.e., for d = 16 − 2.5 = 13.5 in. and p_t = 0.002 read A_s = 0.38 sq in. per ft, similarly for p_t = 0.001 read A_s = 0.19 sq in. per ft.

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN
WORKING STRESS DESIGN
ALLOWABLE STRESSES

ALLOWABLE WORKING STRESSES IN CONCRETE						
Description (Stresses in psi)		Class of Concrete				
		Any	5000	4000	3000	2500
Compressive strength	f_c'	f_c'	5000	4000	3000	2500
Modular ratio, for concrete weighing 145 pcf *	n	$\frac{503.3}{\sqrt{f_c'}}$	7	8	9	10
Flexure:						
Extreme fiber stress in compression	f_c	$0.40f_c'$	2000	1600	1200	1000
Extreme fiber stress in tension in plain concrete	f_t	$1.6\sqrt{f_c'}$	113	102	88	80
Shear (computed as V/bd as a measure of diagonal tension):						
Beams - shear at a distance [d] from the face of the support:						
Beams with no web reinforcement	v_c	$1.1\sqrt{f_c'}$	78	70	60	55
Beams with properly designed web reinforcement	v	$5.0\sqrt{f_c'}$	354	316	274	250
Slabs and Footings - peripheral shear at a distance [d/2] from the periphery of the area of the concentrated load or reaction with no web reinforcement	v_c	$2.0\sqrt{f_c'}$	141	126	110	100
Flexural Bond (deformed bars, sizes ≤ #11):						
Tension top bars	$\bar{u}$	$3.4\sqrt{f_c'}/D \leq 350$				
All other tension bars	u	$4.8\sqrt{f_c'}/D \leq 500$				
		(D = nominal diameter of bar, inches)				

* n is taken to nearest whole number

ALLOWABLE WORKING STRESSES IN REINFORCEMENT		
Description (Stresses in psi)		Grade of Steel
		40, 50, or 60
In tension:	f_s	20000
In compression:		
flexural members	f_s'	20000
columns	f_s	16000

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - DESIGN SECTION	STANDARD DWG. NO. ES-160 SHEET 1 OF 3 DATE 7-64

Revised 8-80

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN
WORKING STRESS DESIGN
ALLOWABLE BOND STRESSES

Allowable Flexural Bond Stress, psi										
Description	Class of Concrete	Bar Size								
		#3	#4	#5	#6	#7	#8	#9	#10	#11
Tension Top Bars	5000	350	350	350	321	275	241	214	192	175
	4000	350	350	342	287	246	215	191	172	156
	3000	350	350	298	248	213	186	165	149	135
	2500	350	340	272	227	194	170	151	136	124
All Other Tension Bars	5000	500	500	500	454	389	340	302	272	247
	4000	500	500	487	405	347	304	270	243	221
	3000	500	500	422	351	301	263	234	211	192
	2500	500	480	384	320	274	240	213	192	174

Flexural Bond Stress

$$u = \frac{V}{\Sigma o j d}$$

V = total shear at the section under investigation, lbs.

Σo = sum of perimeters of the longitudinal tension bars at the section, inches, if all bars are the same size; for mixed sizes, substitute $4\ A_s/D$, where A_s is total longitudinal tension steel area and D is the largest bar diameter.

j = ratio of distance between resultant compression and tension forces to the depth, d. Usually taken as 7/8 for these computations.

d = effective depth, inches.

Flexural bond stress is not considered in compression.

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN
WORKING STRESS DESIGN
DEVELOPMENT LENGTHS

Development Length, inches										
Description	Class of Concrete	Bar Size								
		#3	#4	#5	#6	#7	#8	#9	#10	#11
Tension Top Bars	5000	12	12	14	17	20	25	32	40	49
	4000	12	12	14	17	21	28	35	45	55
	3000	12	12	14	18	25	32	41	52	64
	2500	12	12	14	20	27	36	45	57	70
All Other Tension Bars	5000	12	12	12	12	14	18	23	29	35
	4000	12	12	12	12	15	20	25	32	40
	3000	12	12	12	13	18	23	29	37	46
	2500	12	12	12	14	19	25	32	41	50
All Compression Bars	5000	8	8	8	9	11	12	14	16	17
	4000	8	8	8	10	11	13	15	16	18
	3000	8	8	9	11	13	15	16	18	20
	2500	8	8	10	12	14	16	18	20	22

Notes:

(a) Development lengths are given for design yield strength. $f_y = 40$ ksi

(b) Tension bars spaced laterally not less than 6 inches on center, and bars with at least 3 inches clear from face of member to first bar may use 0.8 development lengths shown but not less than 12 inches.

(c) See ES-227, sheet 3 of 3 for lapped splice lengths.

STRUCTURAL DESIGN: Reinforced Concrete Design
Strength Design
Lap Splices in Reinforcement

Lapped Splice Lengths, inches												
Grade of Steel	Description	Class of Concrete	Class of Splice	Bar Size								
				#3	#4	#5	#6	#7	#8	#9	#10	#11
40	Tension Top Bars	6000	A	12	15	19	23	27	30	34	38	46
			B	16	16	19	23	27	30	38	48	59
			C	21	21	24	29	34	39	50	63	77
		5000	A	12	15	19	23	27	30	34	41	50
			B	16	16	19	23	27	33	42	53	65
			C	21	21	24	29	34	43	54	69	85
		4000	A	12	15	19	23	27	30	36	45	56
			B	16	16	19	23	28	37	47	59	72
			C	21	21	24	29	37	48	61	77	95
		3000	A	12	15	19	23	27	33	41	52	64
			B	16	16	19	24	32	43	54	68	83
			C	21	21	24	31	42	55	70	89	109
		2500	A	12	15	19	23	27	36	45	57	70
			B	16	16	19	26	35	47	59	74	91
			C	21	21	24	34	46	61	77	97	119
	All Other Tension Bars	6000	A	12	15	19	23	27	30	34	38	42
			B	16	16	19	23	27	30	34	38	42
			C	21	21	21	23	27	30	36	45	55
		5000	A	12	15	19	23	27	30	34	38	42
			B	16	16	19	23	27	30	34	38	46
			C	21	21	21	23	27	31	39	49	61
		4000	A	12	15	19	23	27	30	34	38	42
			B	16	16	19	23	27	30	34	42	52
			C	21	21	21	23	27	34	44	55	68
		3000	A	12	15	19	23	27	30	34	38	46
			B	16	16	19	23	27	31	38	49	60
			C	21	21	21	23	30	40	50	64	78
		2500	A	12	15	19	23	27	30	34	41	50
			B	16	16	19	23	27	33	42	53	65
			C	21	21	21	24	33	44	55	70	85
	All Compression Bars	6000		12	12	15	18	21	24	27	30	33
		5000		12	12	15	18	21	24	27	30	33
		4000		12	12	15	18	21	24	27	30	33
		3000		12	12	15	18	21	24	27	30	33
		2500		16	16	20	24	28	32	36	40	44

Tension bars spaced laterally not less than 6 inches on center, and bars with at least 3 inches clear from face of member to first bar may use 0.8 lap lengths shown but not less than 12 inches.

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN
BENDING ONLY OR BENDING COMBINED WITH DIRECT FORCE.
FOR 12 INCH WIDTH

$f'_c = 2500$ psi

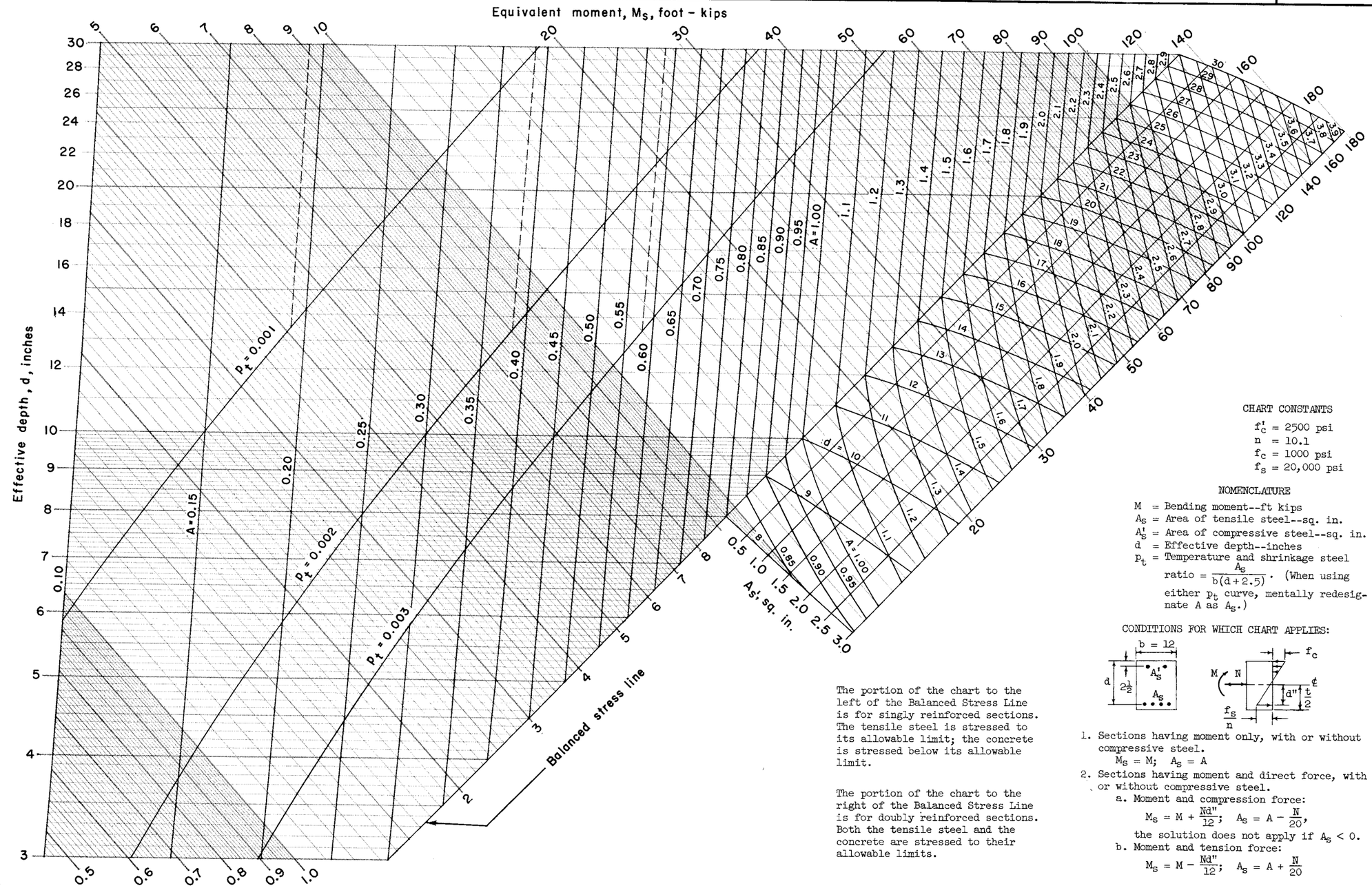


CHART CONSTANTS

$f'_c = 2500$ psi
$n = 10.1$
$f_c = 1000$ psi
$f_s = 20{,}000$ psi

NOMENCLATURE

M = Bending moment--ft kips
A_s = Area of tensile steel--sq. in.
A'_s = Area of compressive steel--sq. in.
d = Effective depth--inches
p_t = Temperature and shrinkage steel ratio $= \frac{A_s}{b(d+2.5)}$. (When using either p_t curve, mentally redesignate A as A_s.)

CONDITIONS FOR WHICH CHART APPLIES:

1. Sections having moment only, with or without compressive steel.
   $M_s = M$; $A_s = A$
2. Sections having moment and direct force, with or without compressive steel.
   a. Moment and compression force:
   $M_s = M + \frac{Nd''}{12}$; $A_s = A - \frac{N}{20}$,
   the solution does not apply if $A_s < 0$.
   b. Moment and tension force:
   $M_s = M - \frac{Nd''}{12}$; $A_s = A + \frac{N}{20}$

The portion of the chart to the left of the Balanced Stress Line is for singly reinforced sections. The tensile steel is stressed to its allowable limit; the concrete is stressed below its allowable limit.

The portion of the chart to the right of the Balanced Stress Line is for doubly reinforced sections. Both the tensile steel and the concrete are stressed to their allowable limits.

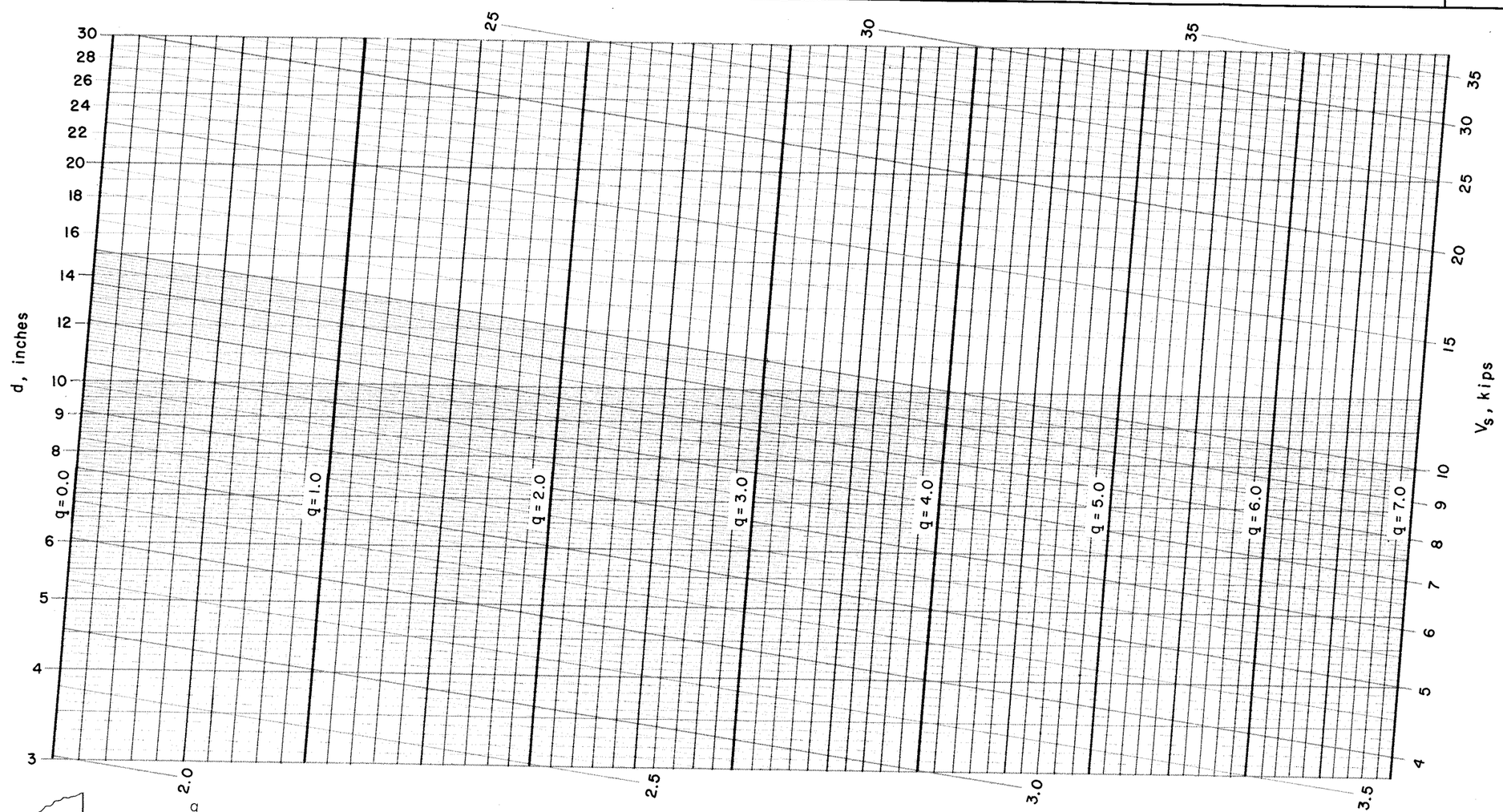
REFERENCE

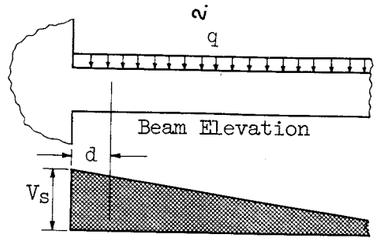
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 161
SHEET 1 OF 3
DATE 4-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
ALLOWABLE TOTAL SHEAR AT FACE OF SUPPORT OF UNIFORMLY LOADED BEAM WITHOUT WEB REINFORCEMENT. FOR 12 INCH WIDTH.

$f'_c = 2500$ psi





$$V_s = \frac{12 v_c d}{1000}\left(1. + \frac{1000q}{144 v_c}\right)$$

V_s = allowable total shear at face of support, kips
v_c = allowable shear stress at distance equal to [d] from face of support = $1.1\sqrt{f'_c}$, psi
d = effective depth, inches
q = unit load, kips per lineal foot

Solution is valid if, within the interval [d]:
(1) there is no concentrated load, and
(2) the unit load is constant.

REFERENCE
ACI Building Code (ACI 318-63)

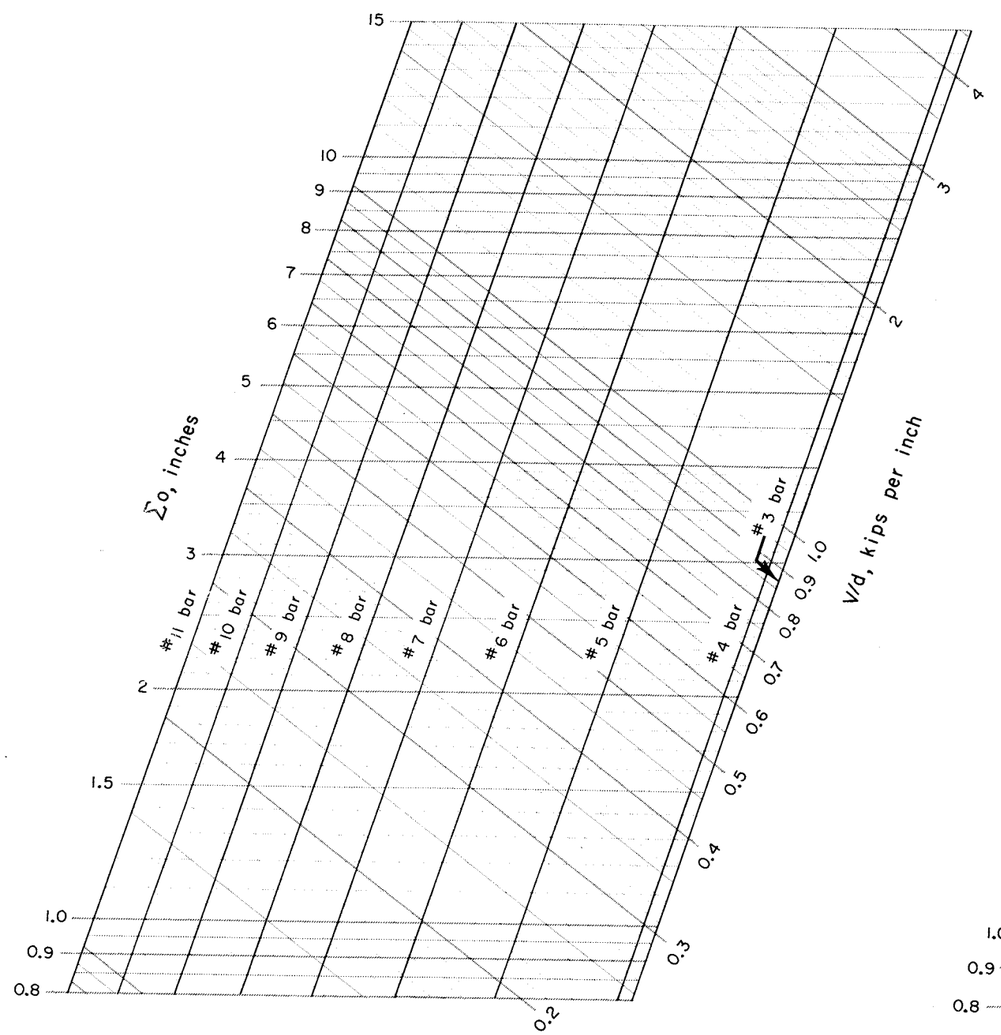
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 161
SHEET 2 OF 3
DATE 7-64

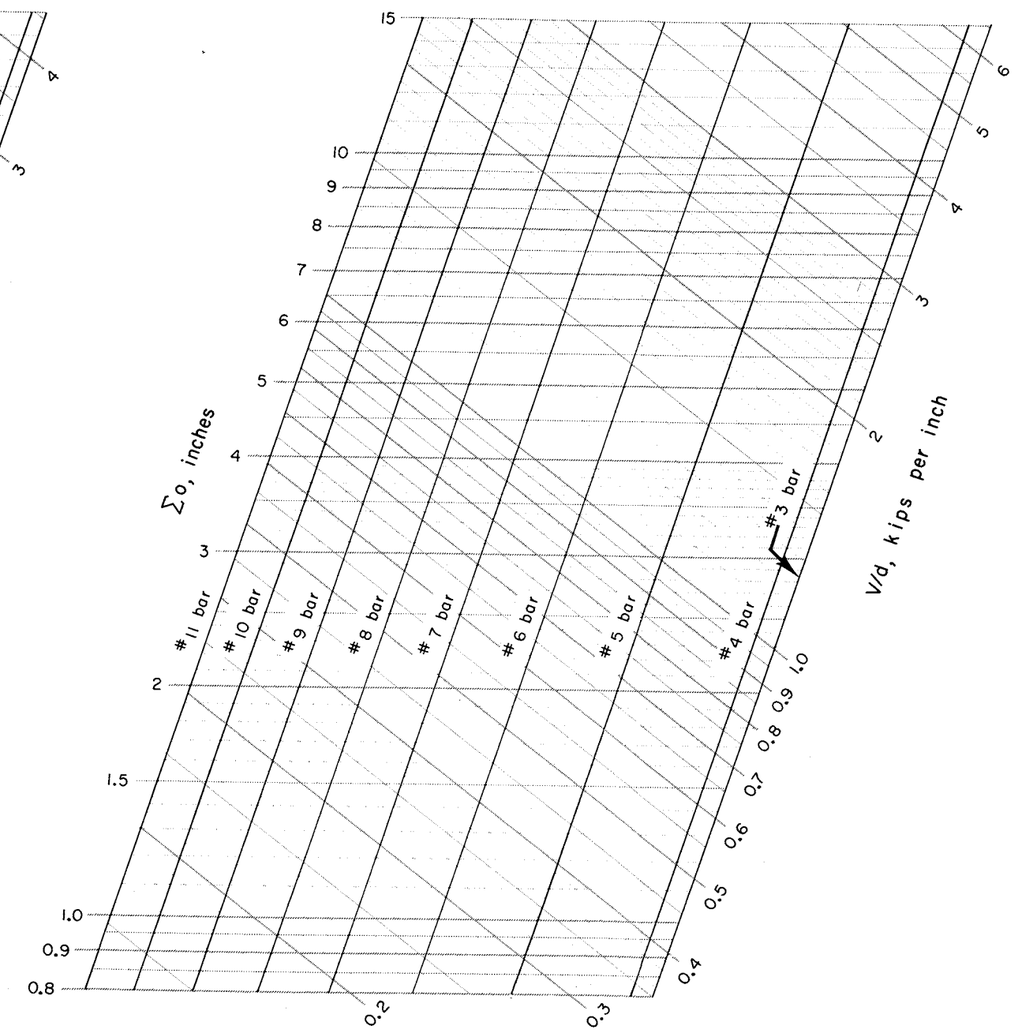
STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
PERIMETER OF TENSILE REINFORCEMENT REQUIRED FOR FLEXURAL BOND.
FOR 12 INCH WIDTH

$f'_c = 2500$ psi

Tension Top Bars



Tension Bars Other Than Top Bars



$$\Sigma o = \frac{8000}{7u}(V/d)$$

Σo = required perimeter of tensile reinforcement at the section, inches
V = total shear at the section, kips
d = effective depth, inches

u = allowable bond stress, psi
For tension top bars $u = 3.4\sqrt{f'_c}/D \leqq 350$
For tension bars other than top bars $u = 4.8\sqrt{f'_c}/D \leqq 500$

REFERENCE

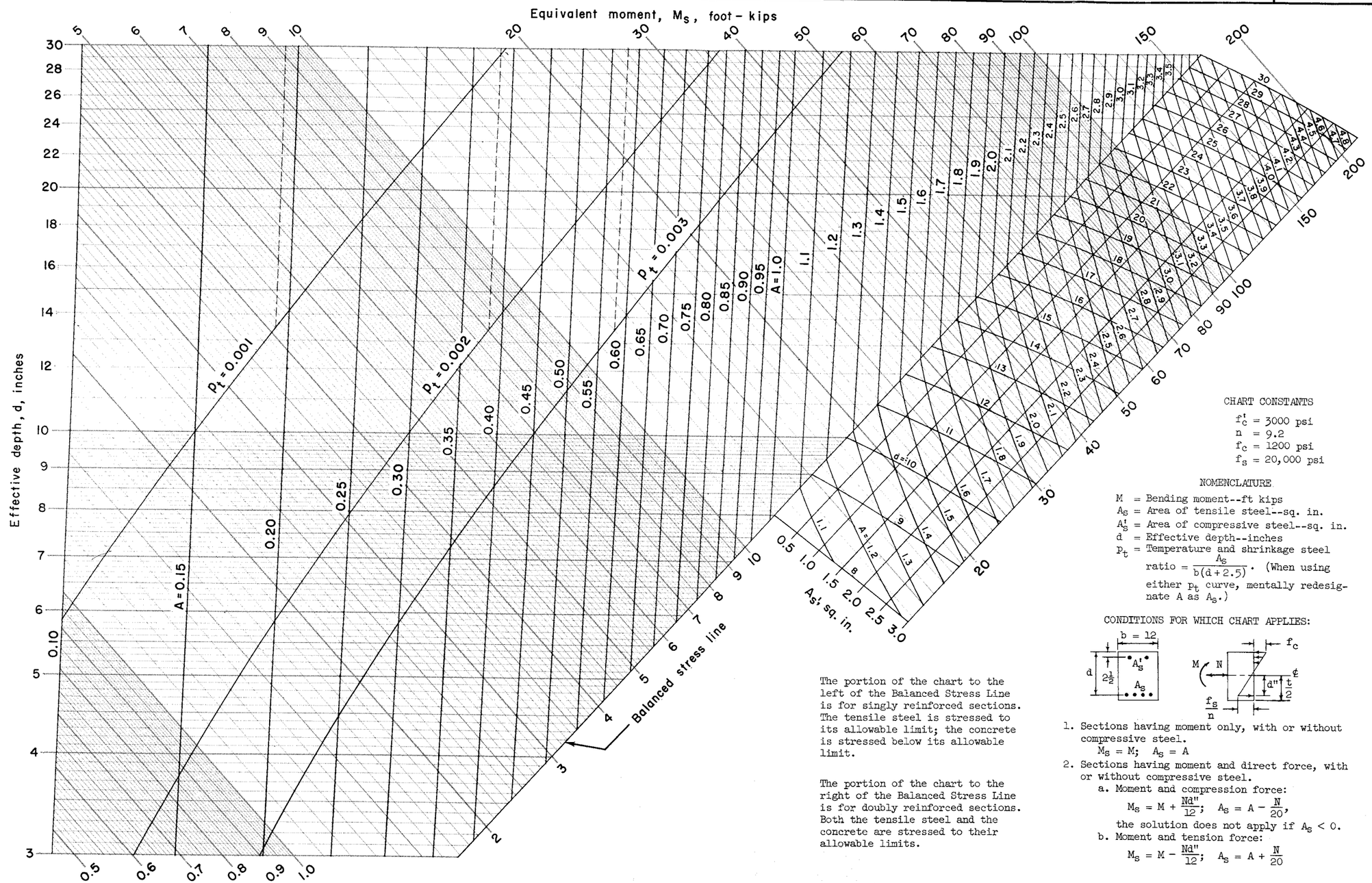
ACI Building Code (ACI 318-63)

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 161
SHEET 3 OF 3
DATE 7-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN BENDING ONLY OR BENDING COMBINED WITH DIRECT FORCE. FOR 12 INCH WIDTH

$f'_c = 3000$ psi



The portion of the chart to the left of the Balanced Stress Line is for singly reinforced sections. The tensile steel is stressed to its allowable limit; the concrete is stressed below its allowable limit.

The portion of the chart to the right of the Balanced Stress Line is for doubly reinforced sections. Both the tensile steel and the concrete are stressed to their allowable limits.

CHART CONSTANTS

$f'_c = 3000$ psi
$n = 9.2$
$f_c = 1200$ psi
$f_s = 20{,}000$ psi

NOMENCLATURE

M = Bending moment--ft kips
A_s = Area of tensile steel--sq. in.
A'_s = Area of compressive steel--sq. in.
d = Effective depth--inches
p_t = Temperature and shrinkage steel ratio $= \frac{A_s}{b(d+2.5)}$. (When using either p_t curve, mentally redesignate A as A_s.)

CONDITIONS FOR WHICH CHART APPLIES:

1. Sections having moment only, with or without compressive steel.
   $M_s = M$; $A_s = A$
2. Sections having moment and direct force, with or without compressive steel.
   a. Moment and compression force:
   $M_s = M + \frac{Nd''}{12}$; $A_s = A - \frac{N}{20}$,
   the solution does not apply if $A_s < 0$.
   b. Moment and tension force:
   $M_s = M - \frac{Nd''}{12}$; $A_s = A + \frac{N}{20}$

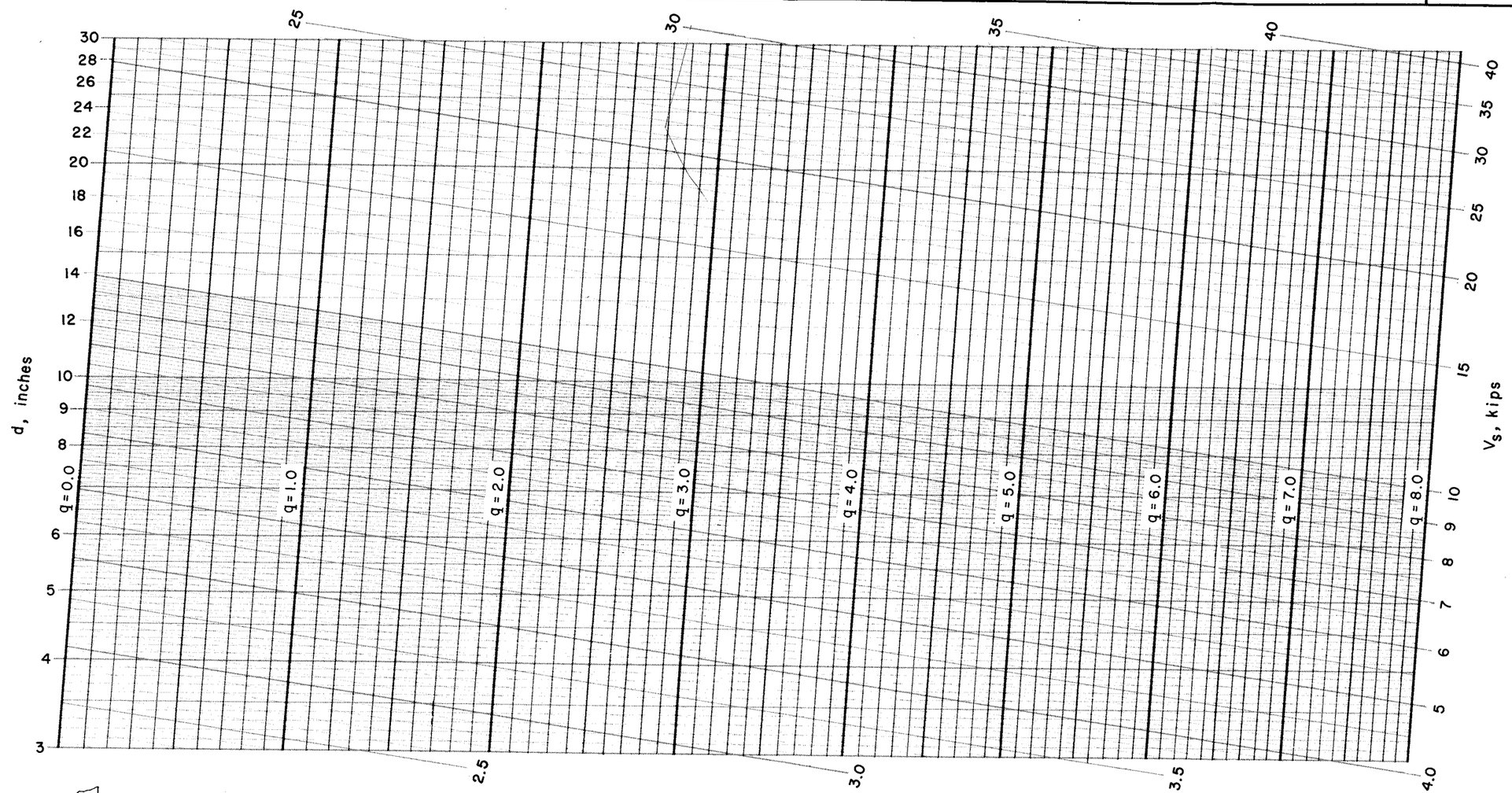
REFERENCE

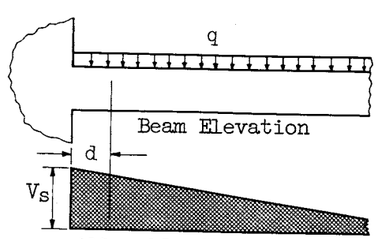
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 162
SHEET 1 OF 3
DATE 4-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
ALLOWABLE TOTAL SHEAR AT FACE OF SUPPORT OF UNIFORMLY LOADED BEAM WITHOUT WEB REINFORCEMENT. FOR 12 INCH WIDTH.

$f'_c = 3000$ psi





Allowable Shear Diagram

$$V_s = \frac{12 v_c d}{1000}\left(1 + \frac{1000q}{144 v_c}\right)$$

V_s = allowable total shear at face of support, kips
v_c = allowable shear stress at distance equal to [d] from face of support = $1.1\sqrt{f'_c}$, psi
d = effective depth, inches
q = unit load, kips per lineal foot

Solution is valid if, within the interval [d]:
(1) there is no concentrated load, and
(2) the unit load is constant.

REFERENCE
ACI Building Code (ACI 318-63)

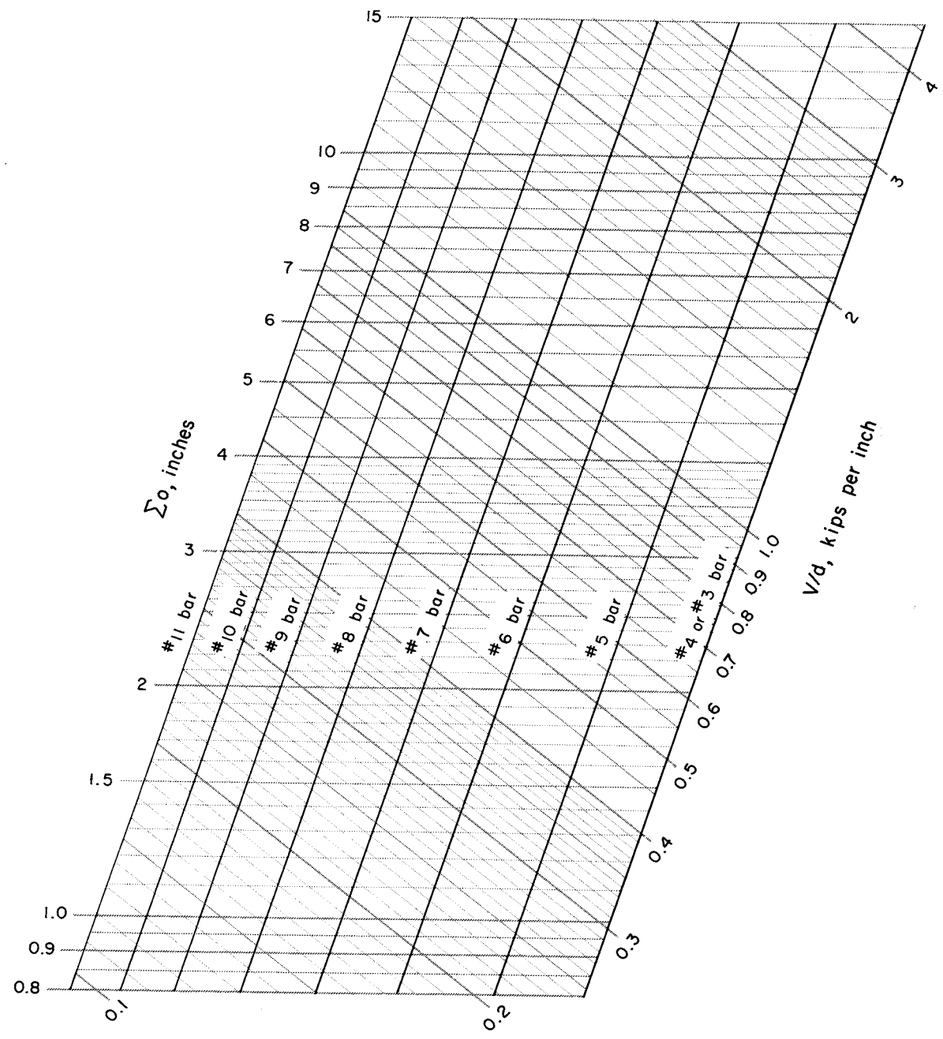
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 162
SHEET 2 OF 3
DATE 7-64

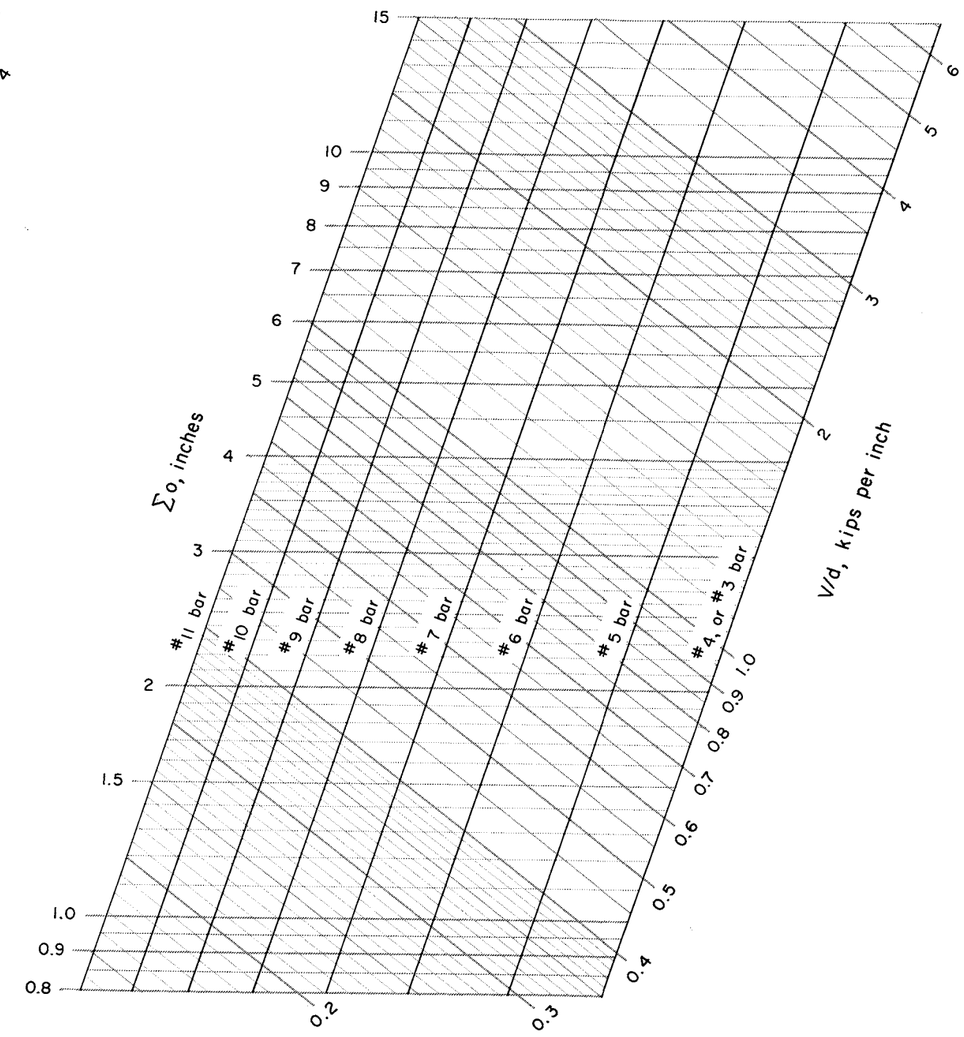
STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
PERIMETER OF TENSILE REINFORCEMENT REQUIRED FOR FLEXURAL BOND.
FOR 12 INCH WIDTH

$f'_c = 3000$ psi

Tension Top Bars



Tension Bars Other Than Top Bars



$$\Sigma o = \frac{8000}{7u}(V/d)$$

Σo = required perimeter of tensile reinforcement at the section, inches
V = total shear at the section, kips
d = effective depth, inches

u = allowable bond stress, psi

For tension top bars $u = 3.4\sqrt{f'_c}/D \leq 350$

For tension bars other than top bars $u = 4.8\sqrt{f'_c}/D \leq 500$

REFERENCE

ACI Building Code (ACI 318 – 63)

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES-162
SHEET 3 OF 3
DATE 7-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN BENDING ONLY OR BENDING COMBINED WITH DIRECT FORCE. FOR 12 INCH WIDTH

$f'_c = 3500$ psi

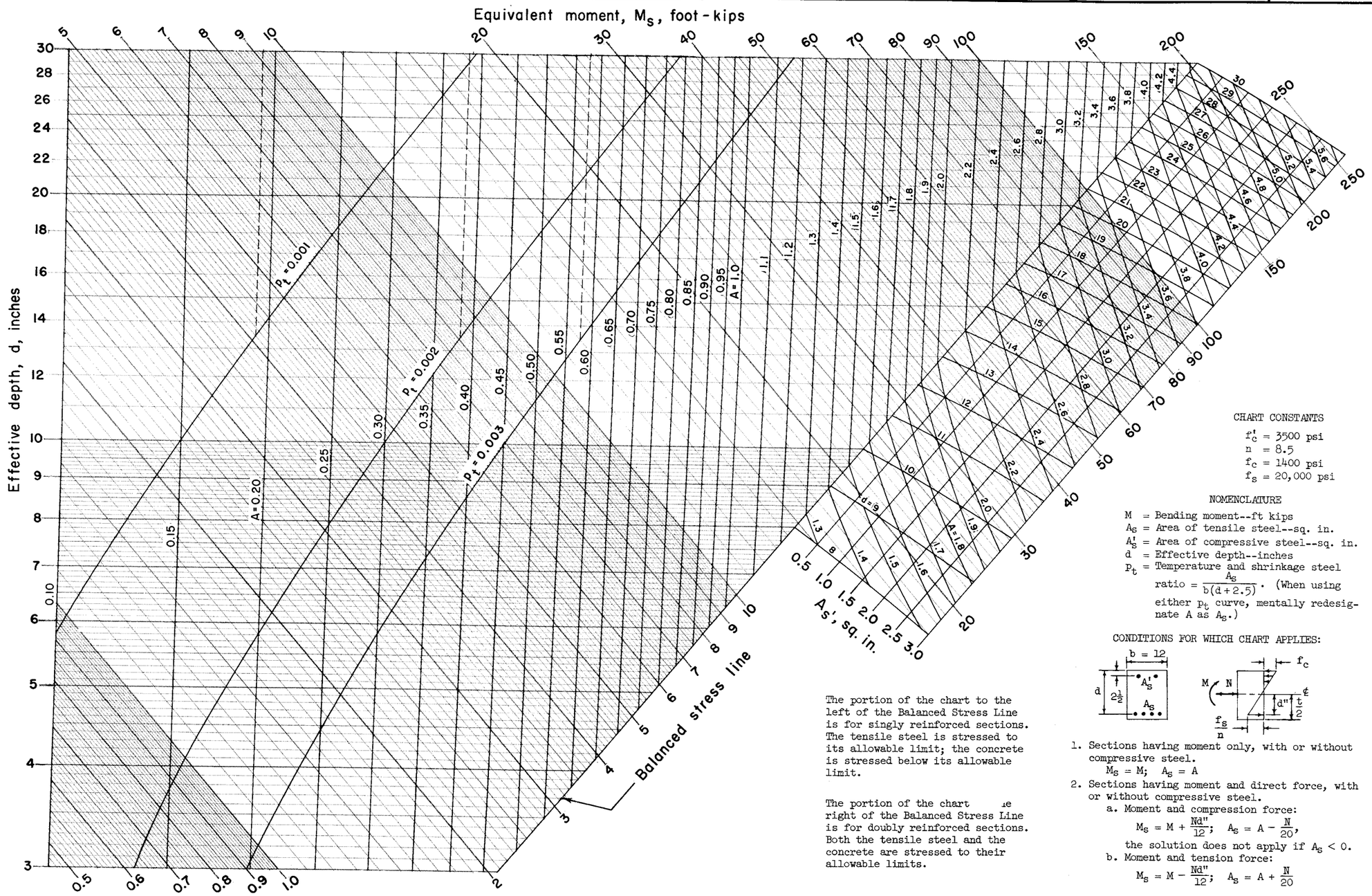


CHART CONSTANTS

$f'_c = 3500$ psi
$n = 8.5$
$f_c = 1400$ psi
$f_s = 20{,}000$ psi

NOMENCLATURE

M = Bending moment--ft kips
A_s = Area of tensile steel--sq. in.
A'_s = Area of compressive steel--sq. in.
d = Effective depth--inches
p_t = Temperature and shrinkage steel ratio $= \frac{A_s}{b(d+2.5)}$. (When using either p_t curve, mentally redesignate A as A_s.)

CONDITIONS FOR WHICH CHART APPLIES:

1. Sections having moment only, with or without compressive steel.
   $M_s = M$; $A_s = A$
2. Sections having moment and direct force, with or without compressive steel.
   a. Moment and compression force:
   $M_s = M + \frac{Nd''}{12}$; $A_s = A - \frac{N}{20}$,
   the solution does not apply if $A_s < 0$.
   b. Moment and tension force:
   $M_s = M - \frac{Nd''}{12}$; $A_s = A + \frac{N}{20}$

The portion of the chart to the left of the Balanced Stress Line is for singly reinforced sections. The tensile steel is stressed to its allowable limit; the concrete is stressed below its allowable limit.

The portion of the chart [illegible] right of the Balanced Stress Line is for doubly reinforced sections. Both the tensile steel and the concrete are stressed to their allowable limits.

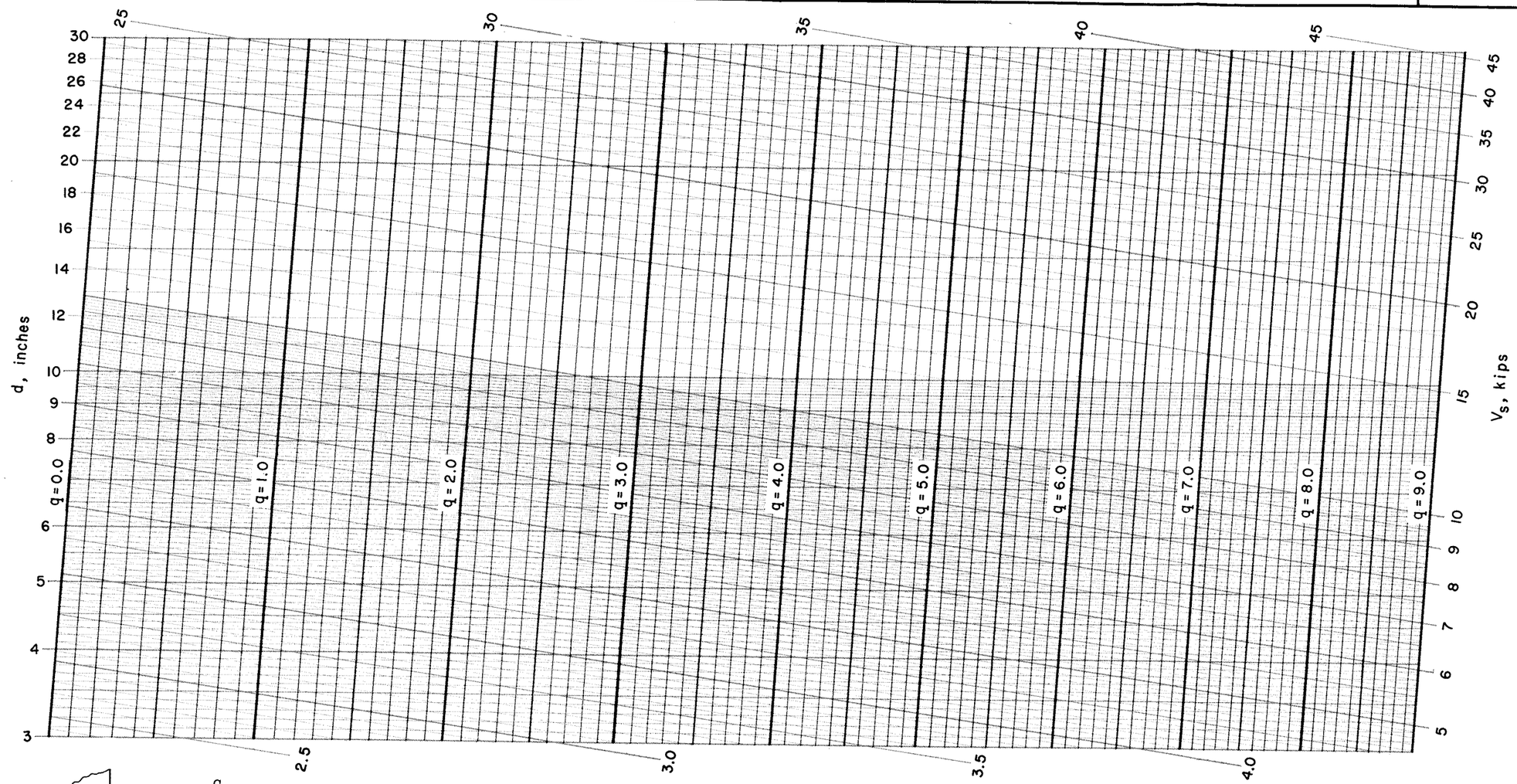
REFERENCE

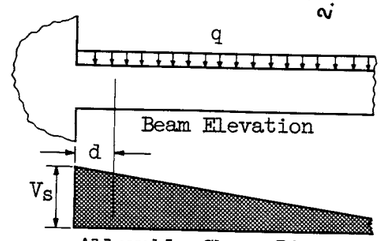
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO. ES- 163
SHEET 1 OF 3
DATE 4-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
ALLOWABLE TOTAL SHEAR AT FACE OF SUPPORT OF UNIFORMLY LOADED
BEAM WITHOUT WEB REINFORCEMENT. FOR 12 INCH WIDTH.

$f'_c = 3500$ psi





Allowable Shear Diagram

$$V_s = \frac{12 v_c d}{1000}\left(1 + \frac{1000q}{144 v_c}\right)$$

V_s = allowable total shear at face of support, kips
v_c = allowable shear stress at distance equal to [d] from face of support = $1.1\sqrt{f'_c}$, psi
d = effective depth, inches
q = unit load, kips per lineal foot

Solution is valid if, within the interval [d]:
(1) there is no concentrated load, and
(2) the unit load is constant.

REFERENCE
ACI Building Code (ACI 318-63)

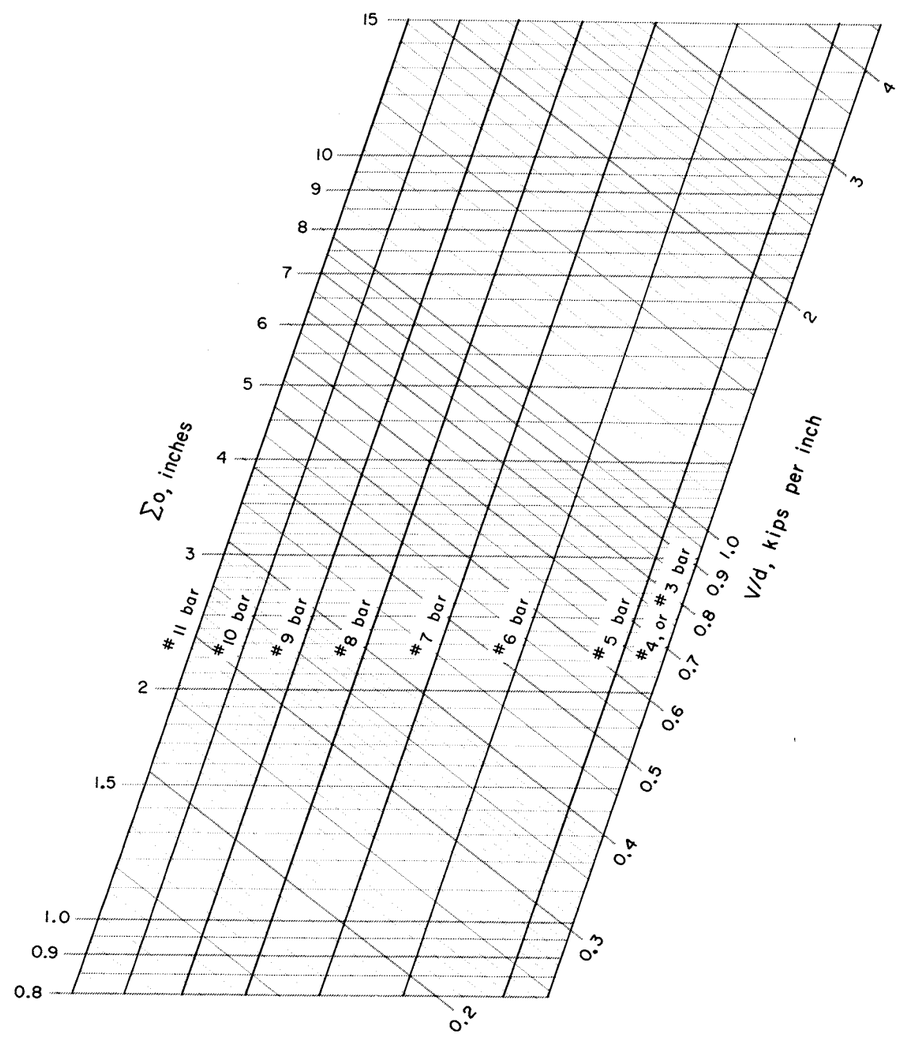
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 163
SHEET 2 OF 3
DATE 7 - 64

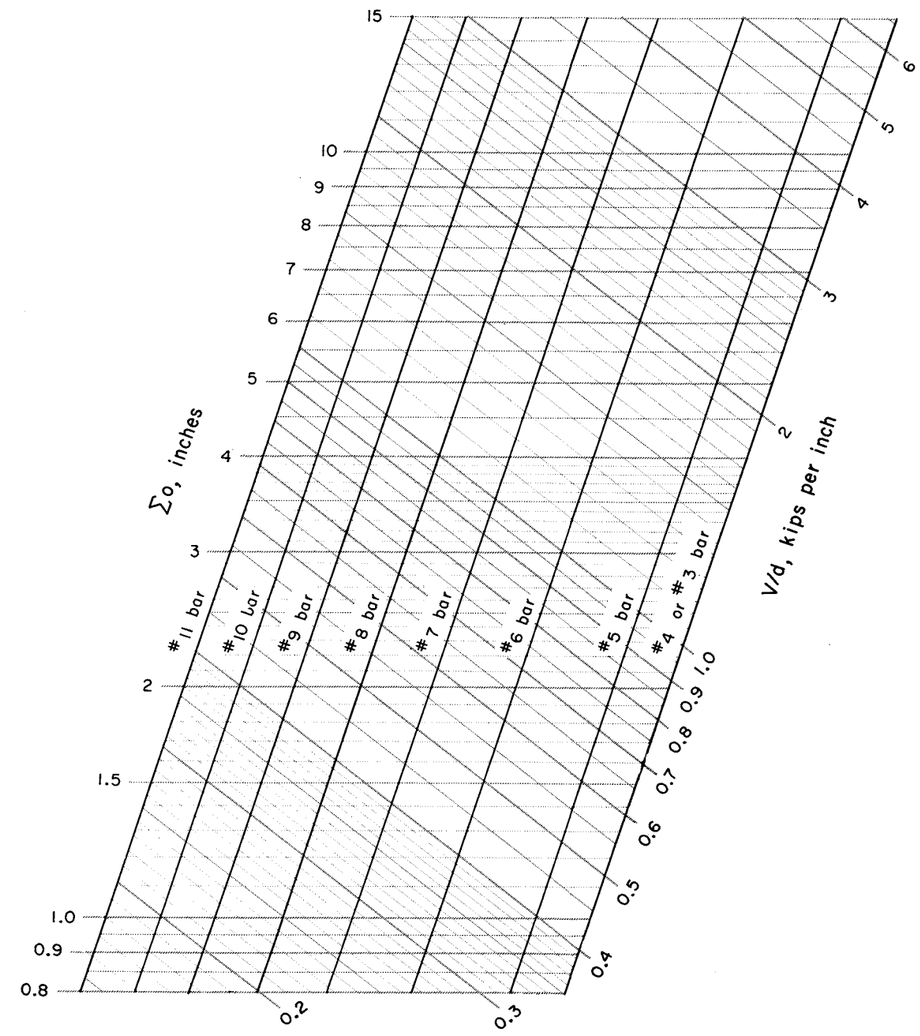
STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
PERIMETER OF TENSILE REINFORCEMENT REQUIRED FOR FLEXURAL BOND.
FOR 12 INCH WIDTH

$f'_c = 3500$ psi

Tension Top Bars



Tension Bars Other Than Top Bars



$$\Sigma o = \frac{8000}{7u}(V/d)$$

Σo = required perimeter of tensile reinforcement at the section, inches
V = total shear at the section, kips
d = effective depth, inches

u = allowable bond stress, psi
For tension top bars $u = 3.4\sqrt{f'_c}/D \leq 350$
For tension bars other than top bars $u = 4.8\sqrt{f'_c}/D \leq 500$

REFERENCE

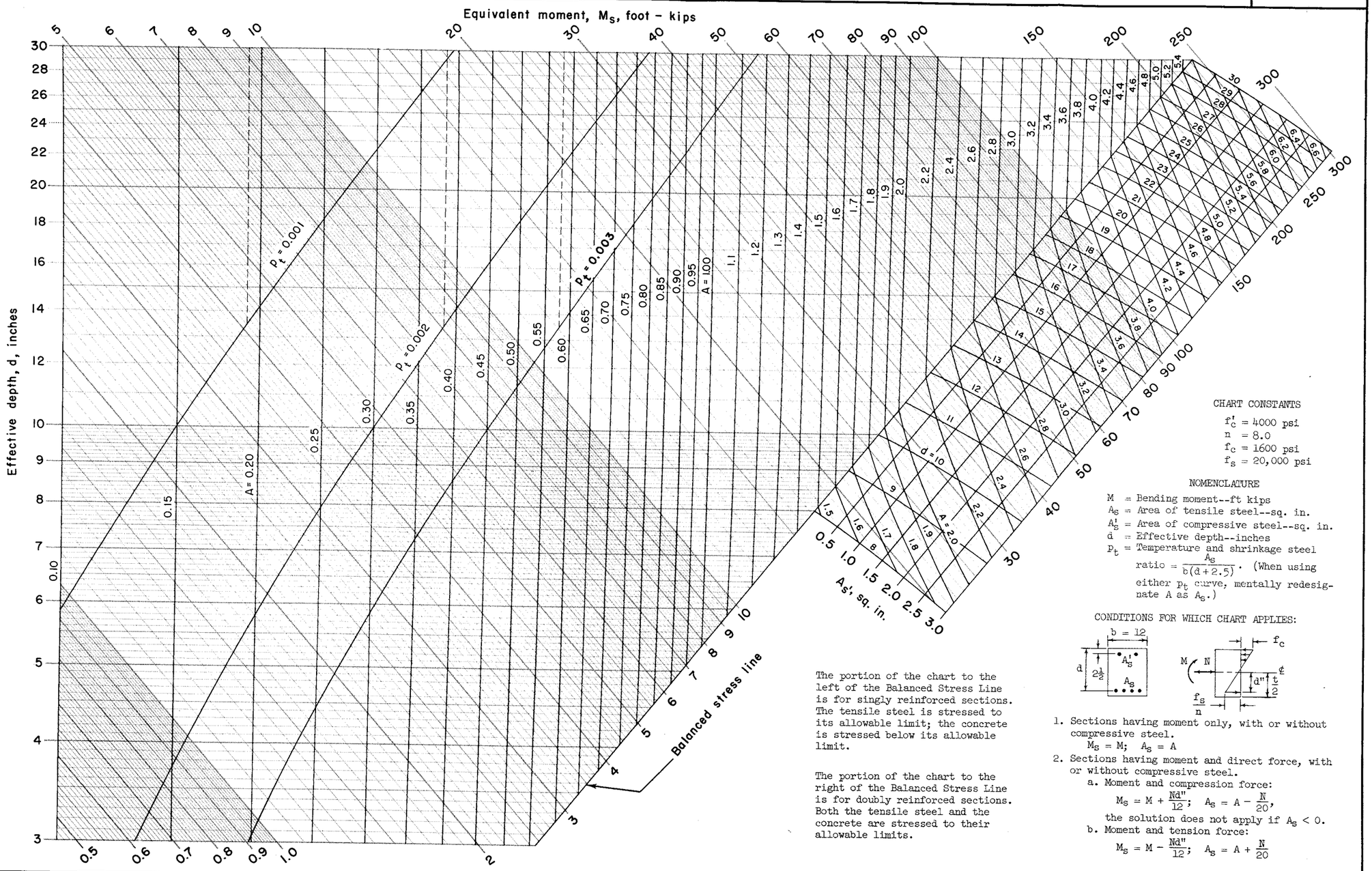
ACI Building Code (ACI 318 - 63)

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES-163
SHEET 3 OF 3
DATE 7-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN
BENDING ONLY OR BENDING COMBINED WITH DIRECT FORCE.
FOR 12 INCH WIDTH

f'_c = 4000 psi



The portion of the chart to the left of the Balanced Stress Line is for singly reinforced sections. The tensile steel is stressed to its allowable limit; the concrete is stressed below its allowable limit.

The portion of the chart to the right of the Balanced Stress Line is for doubly reinforced sections. Both the tensile steel and the concrete are stressed to their allowable limits.

CHART CONSTANTS

f'_c = 4000 psi
n = 8.0
f_c = 1600 psi
f_s = 20,000 psi

NOMENCLATURE

M = Bending moment--ft kips
A_s = Area of tensile steel--sq. in.
A'_s = Area of compressive steel--sq. in.
d = Effective depth--inches
p_t = Temperature and shrinkage steel ratio = $\frac{A_s}{b(d+2.5)}$. (When using either p_t curve, mentally redesignate A as A_s.)

CONDITIONS FOR WHICH CHART APPLIES:

1. Sections having moment only, with or without compressive steel.
   $M_s = M$; $A_s = A$
2. Sections having moment and direct force, with or without compressive steel.
   a. Moment and compression force:
   $M_s = M + \frac{Nd''}{12}$; $A_s = A - \frac{N}{20}$,
   the solution does not apply if $A_s < 0$.
   b. Moment and tension force:
   $M_s = M - \frac{Nd''}{12}$; $A_s = A + \frac{N}{20}$

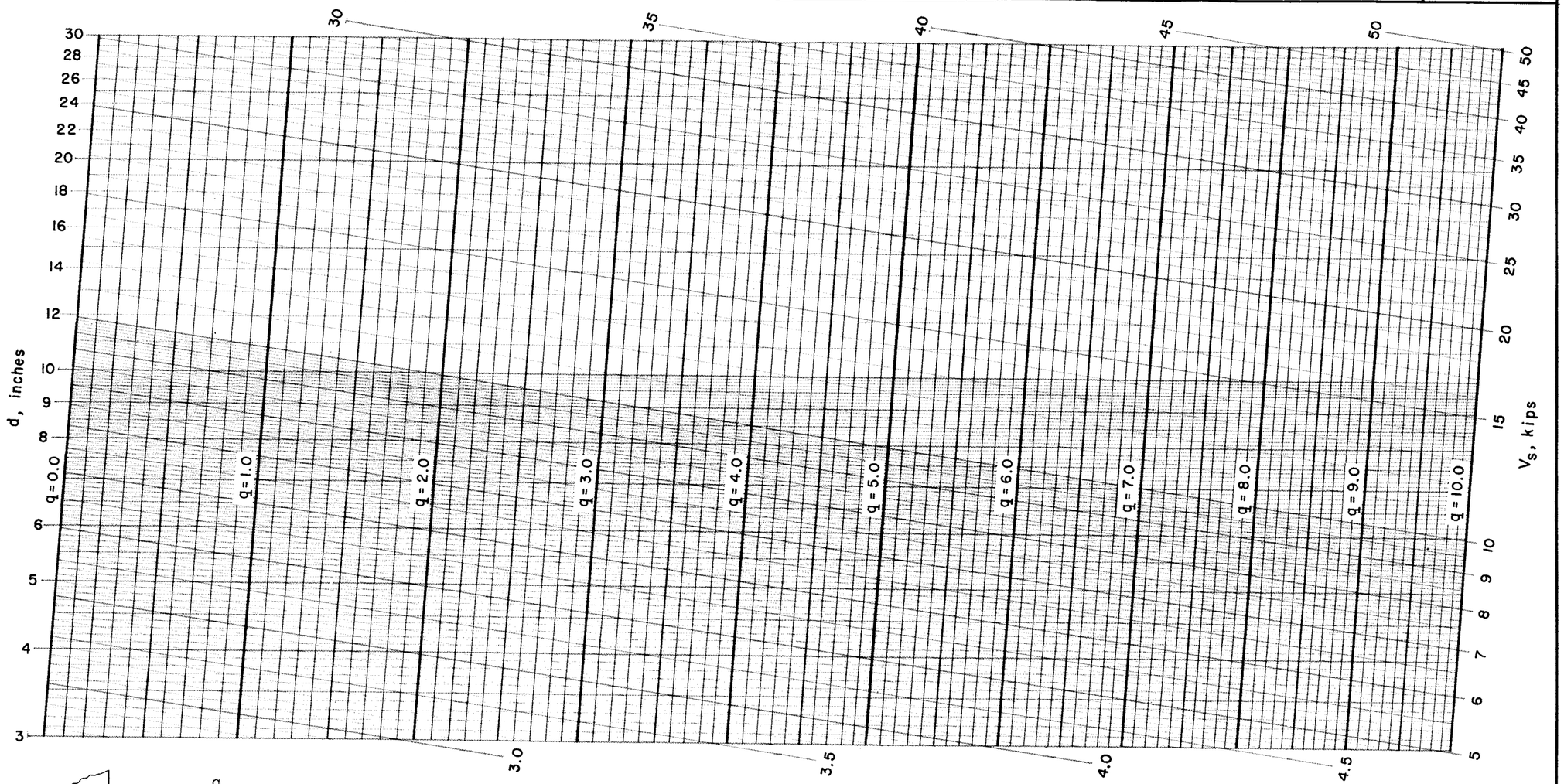
REFERENCE

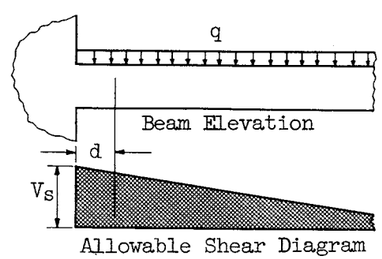
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 164
SHEET 1 OF 3
DATE 4-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
ALLOWABLE TOTAL SHEAR AT FACE OF SUPPORT OF UNIFORMLY LOADED BEAM WITHOUT WEB REINFORCEMENT. FOR 12 INCH WIDTH.

$f'_c = 4000$ psi





$$V_s = \frac{12 v_c d}{1000}\left(1 + \frac{1000q}{144 v_c}\right)$$

V_s = allowable total shear at face of support, kips
v_c = allowable shear stress at distance equal to [d] from face of support = $1.1\sqrt{f'_c}$, psi
d = effective depth, inches
q = unit load, kips per lineal foot

Solution is valid if, within the interval [d]:
(1) there is no concentrated load, and
(2) the unit load is constant.

REFERENCE
ACI Building Code (ACI 318-63)

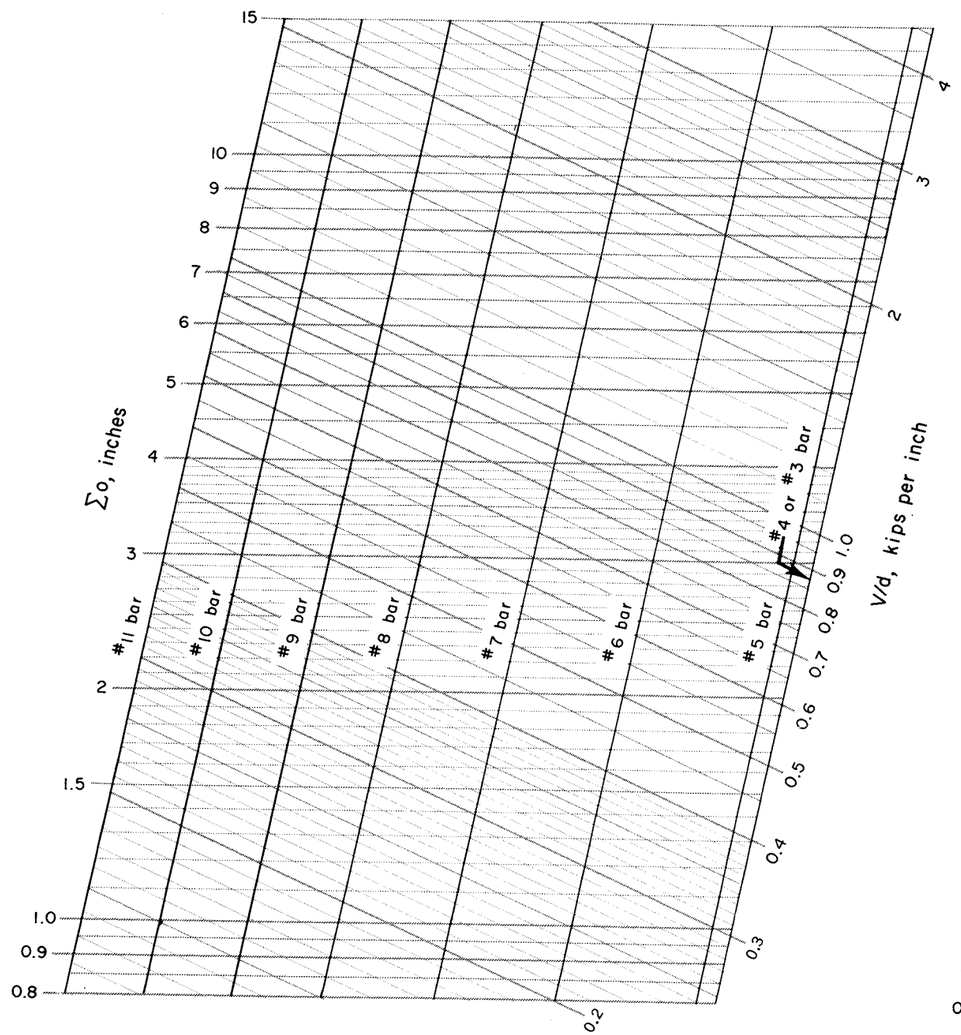
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 164
SHEET 2 OF 3
DATE 7-64

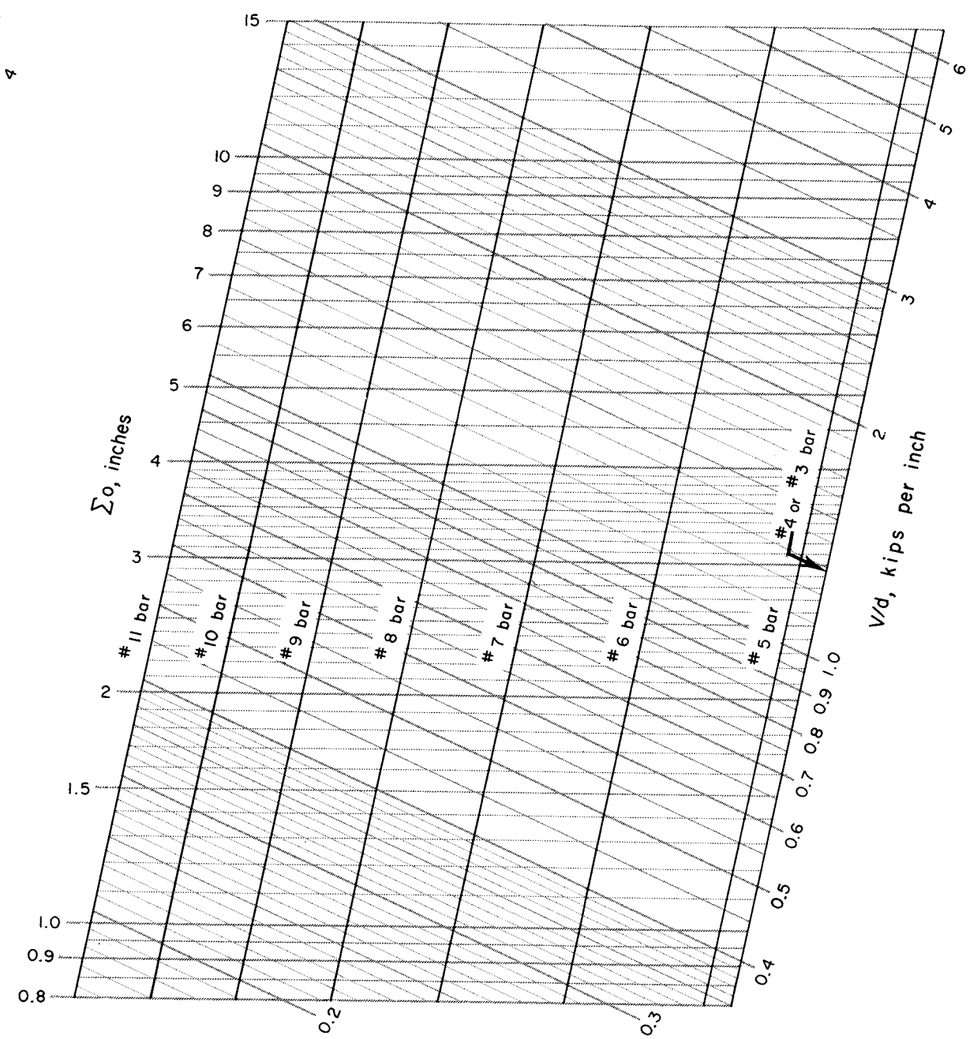
STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
PERIMETER OF TENSILE REINFORCEMENT REQUIRED FOR FLEXURAL BOND.
FOR 12 INCH WIDTH

$f'_c = 4000$ psi

Tension Top Bars



Tension Bars Other Than Top Bars



$$\Sigma o = \frac{8000}{7u}(V/d)$$

Σo = required perimeter of tensile reinforcement at the section, inches
V = total shear at the section, kips
d = effective depth, inches

u = allowable bond stress, psi
For tension top bars $u = 3.4\sqrt{f'_c}/D \leq 350$
For tension bars other than top bars $u = 4.8\sqrt{f'_c}/D \leq 500$

REFERENCE

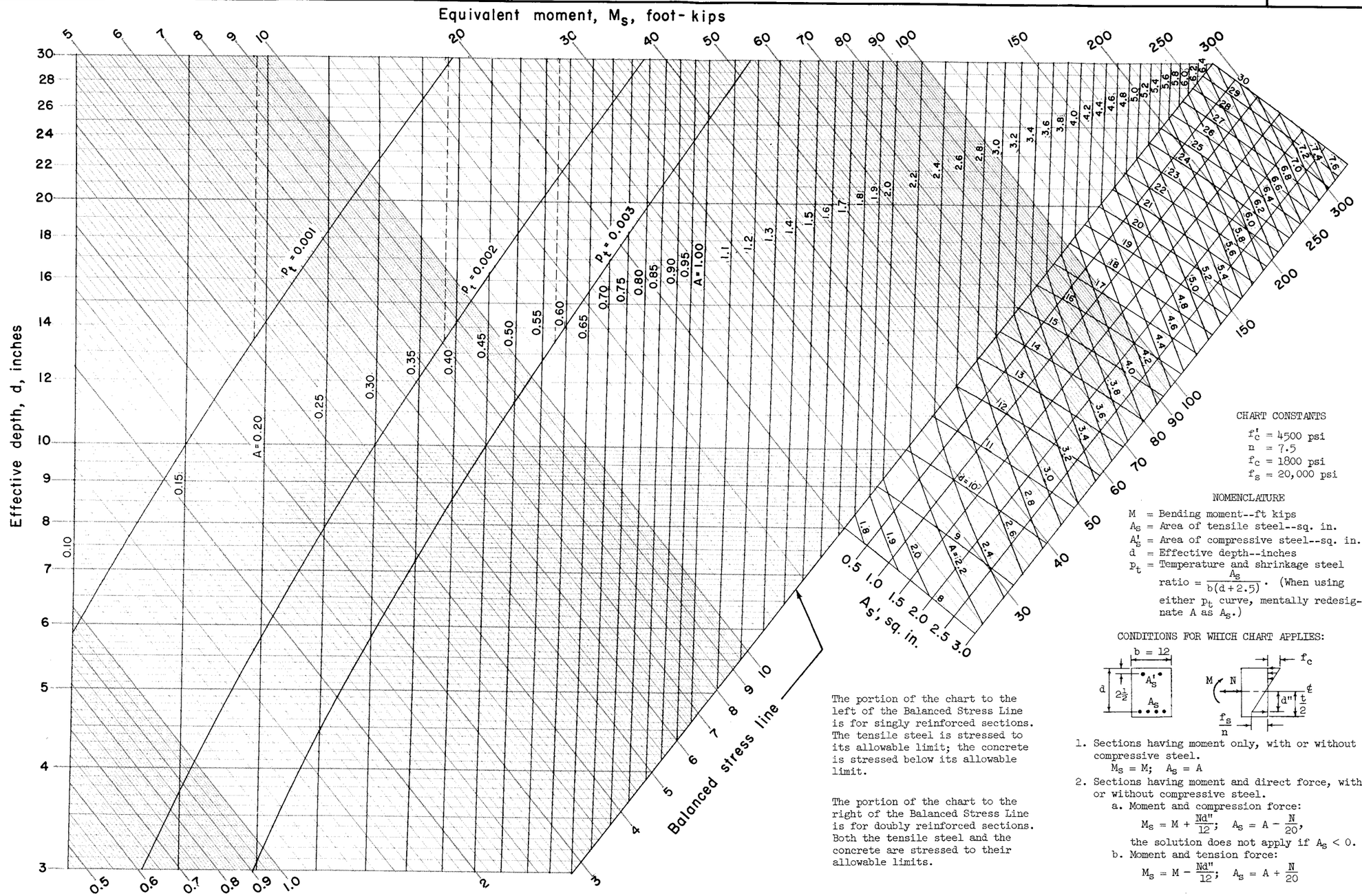
ACI Building Code (ACI 318-63)

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 164
SHEET 3 OF 3
DATE 7-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN
BENDING ONLY OR BENDING COMBINED WITH DIRECT FORCE.
FOR 12 INCH WIDTH

$f'_c = 4500$ psi



The portion of the chart to the left of the Balanced Stress Line is for singly reinforced sections. The tensile steel is stressed to its allowable limit; the concrete is stressed below its allowable limit.

The portion of the chart to the right of the Balanced Stress Line is for doubly reinforced sections. Both the tensile steel and the concrete are stressed to their allowable limits.

CHART CONSTANTS

$f'_c = 4500$ psi
$n = 7.5$
$f_c = 1800$ psi
$f_s = 20{,}000$ psi

NOMENCLATURE

M = Bending moment--ft kips
A_s = Area of tensile steel--sq. in.
A'_s = Area of compressive steel--sq. in.
d = Effective depth--inches
p_t = Temperature and shrinkage steel ratio $= \frac{A_s}{b(d+2.5)}$. (When using either p_t curve, mentally redesignate A as A_s.)

CONDITIONS FOR WHICH CHART APPLIES:

1. Sections having moment only, with or without compressive steel.
   $M_s = M$; $A_s = A$
2. Sections having moment and direct force, with or without compressive steel.
   a. Moment and compression force:
   $M_s = M + \frac{Nd''}{12}$; $A_s = A - \frac{N}{20}$,
   the solution does not apply if $A_s < 0$.
   b. Moment and tension force:
   $M_s = M - \frac{Nd''}{12}$; $A_s = A + \frac{N}{20}$

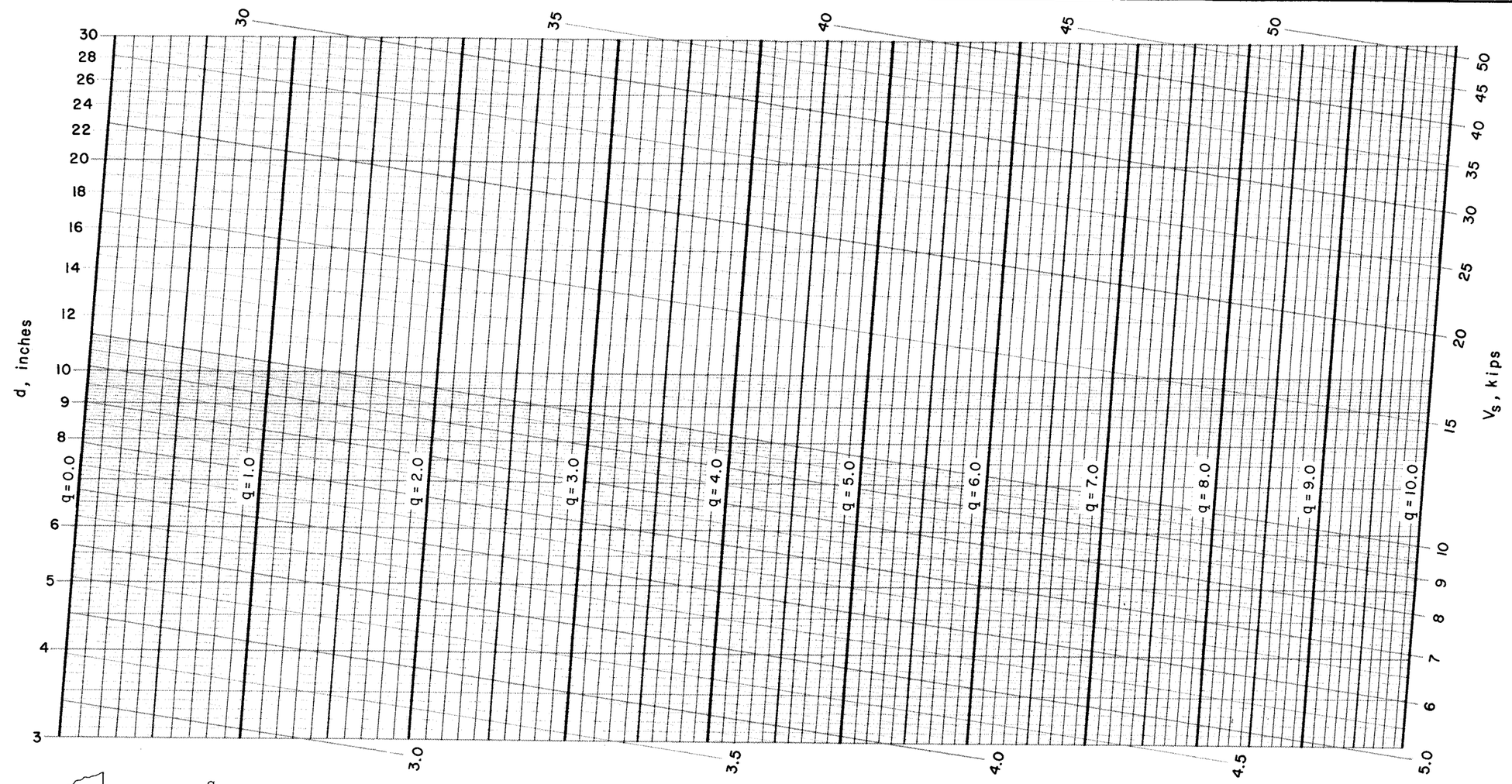
REFERENCE

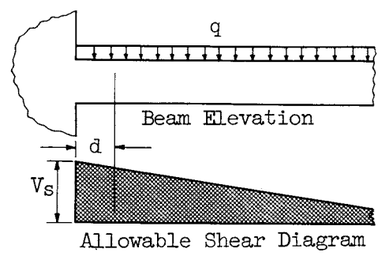
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 165
SHEET 1 OF 3
DATE 4-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
ALLOWABLE TOTAL SHEAR AT FACE OF SUPPORT OF UNIFORMLY LOADED BEAM WITHOUT WEB REINFORCEMENT. FOR 12 INCH WIDTH.

$f'_c = 4500$ psi





Allowable Shear Diagram

$$V_s = \frac{12 v_c d}{1000}\left(1 + \frac{1000q}{144 v_c}\right)$$

V_s = allowable total shear at face of support, kips
v_c = allowable shear stress at distance equal to [d] from face of support = $1.1\sqrt{f'_c}$. psi
d = effective depth, inches
q = unit load, kips per lineal foot

Solution is valid if, within the interval [d]:
(1) there is no concentrated load, and
(2) the unit load is constant.

REFERENCE
ACI Building Code (ACI 318-63)

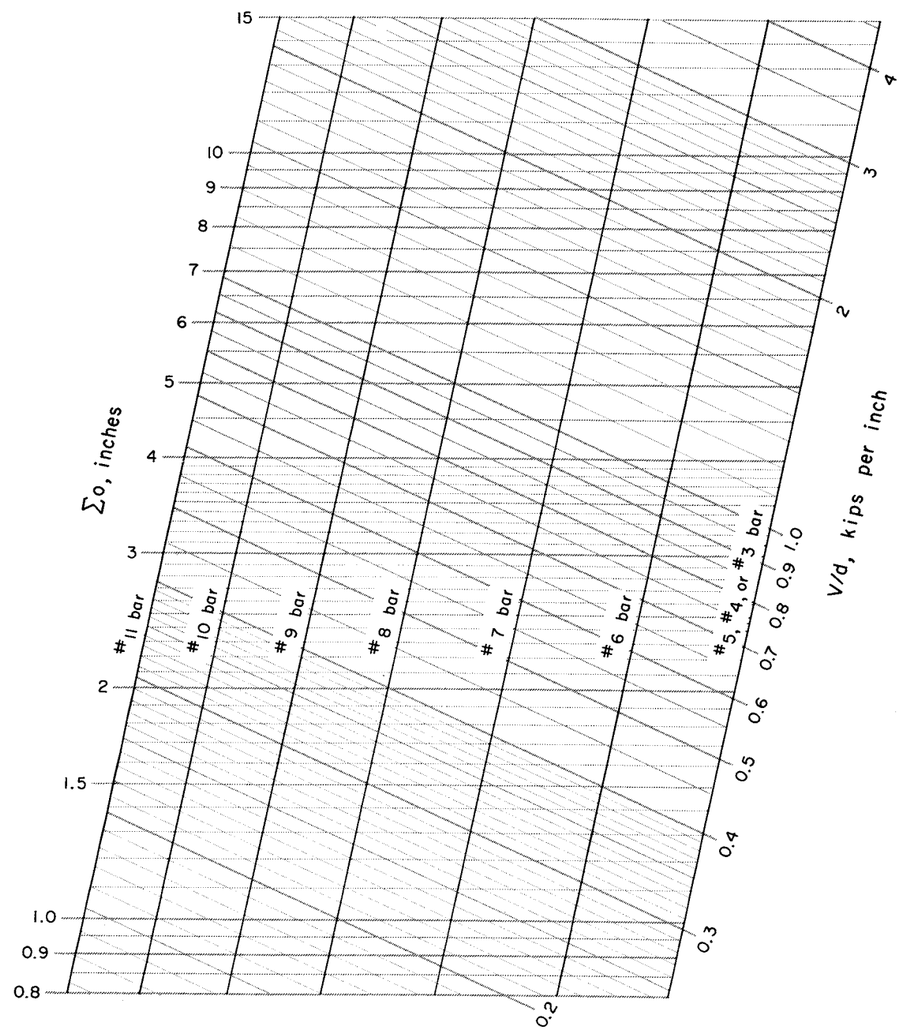
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 165
SHEET 2 OF 3
DATE 7-64

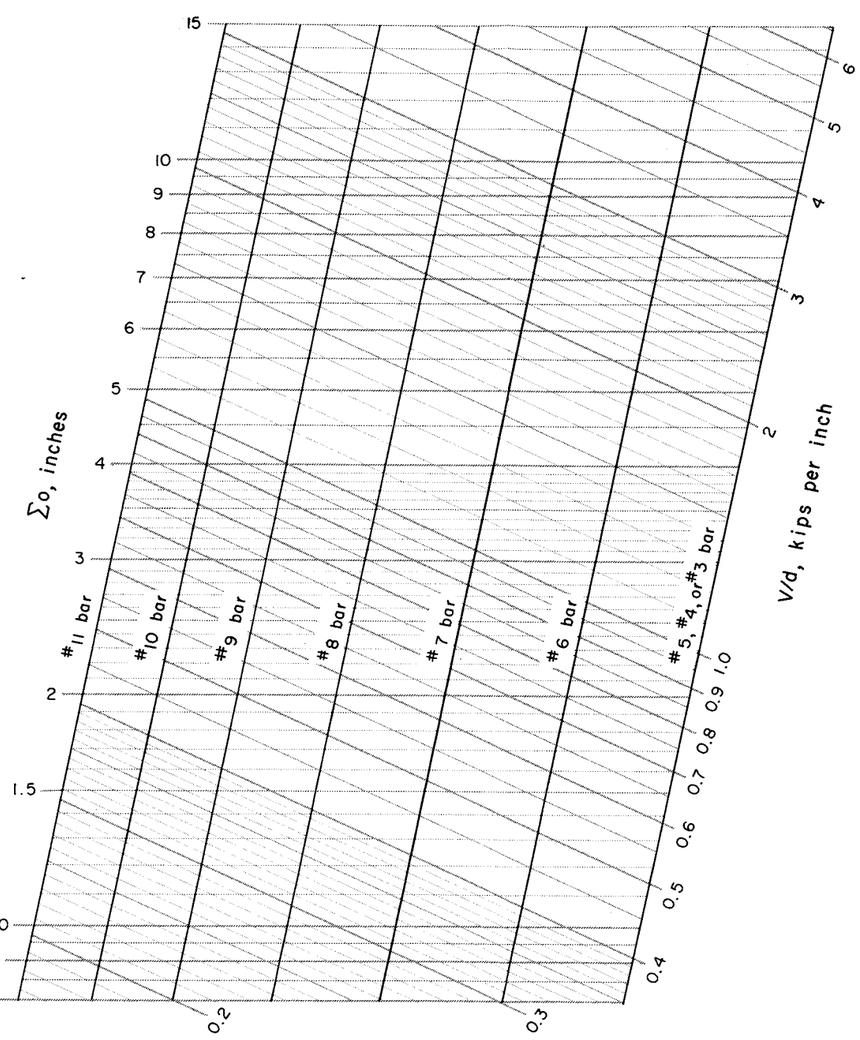
STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
PERIMETER OF TENSILE REINFORCEMENT REQUIRED FOR FLEXURAL BOND.
FOR 12 INCH WIDTH

$f'_c = 4500$ psi

Tension Top Bars



Tension Bars Other Than Top Bars



$$\Sigma o = \frac{8000}{7u}(V/d)$$

Σo = required perimeter of tensile reinforcement at the section, inches
V = total shear at the section, kips
d = effective depth, inches

u = allowable bond stress, psi
For tension top bars $u = 3.4\sqrt{f'_c}/D \leq 350$
For tension bars other than top bars $u = 4.8\sqrt{f'_c}/D \leq 500$

REFERENCE

ACI Building Code (ACI 318 - 63)

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 165
SHEET 3 OF 3
DATE 7-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN
BENDING ONLY OR BENDING COMBINED WITH DIRECT FORCE.
FOR 12 INCH WIDTH

$f'_c = 5000$ psi

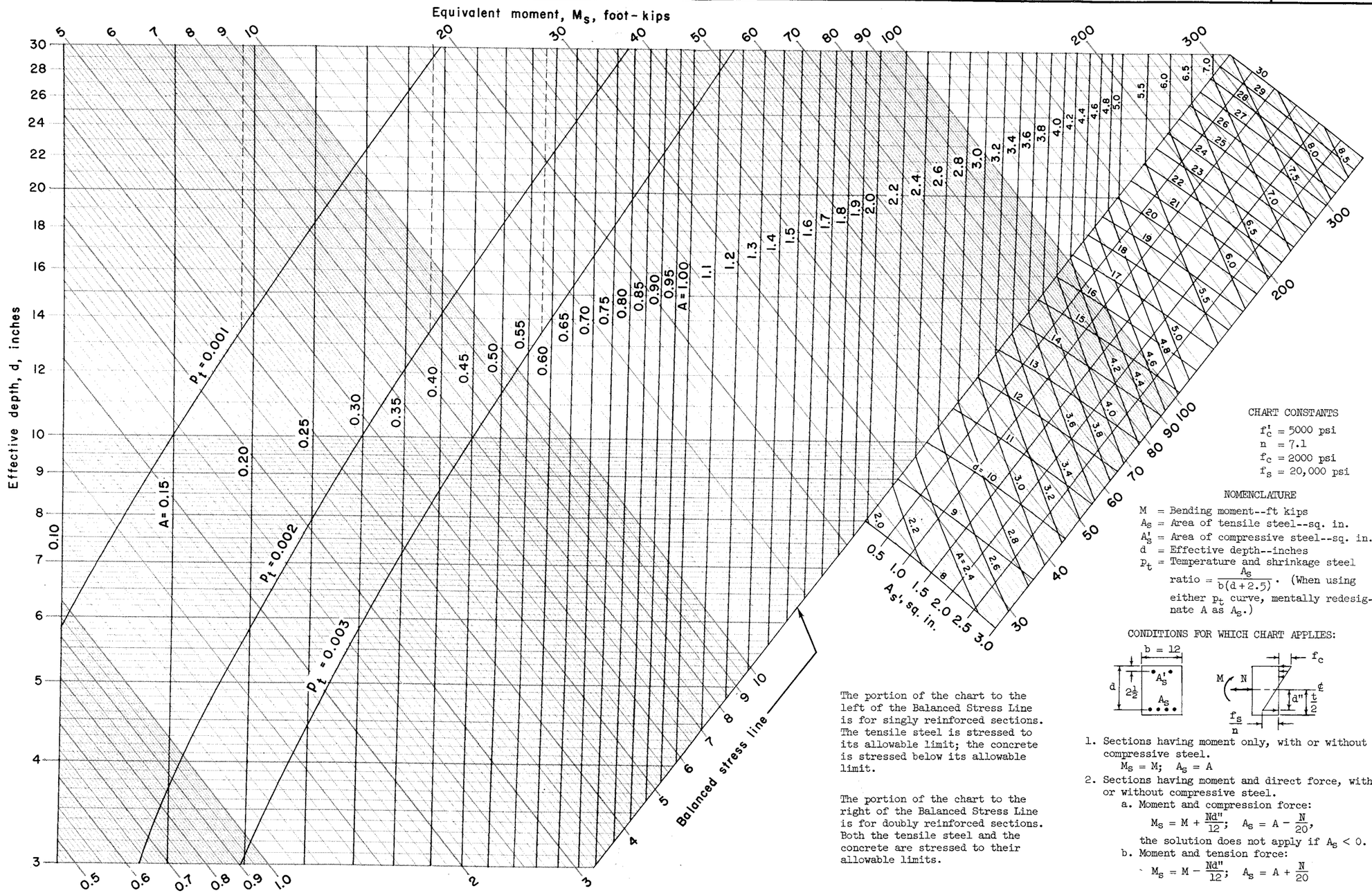


CHART CONSTANTS

$f'_c = 5000$ psi
$n = 7.1$
$f_c = 2000$ psi
$f_s = 20,000$ psi

NOMENCLATURE

M = Bending moment--ft kips
A_s = Area of tensile steel--sq. in.
A'_s = Area of compressive steel--sq. in.
d = Effective depth--inches
p_t = Temperature and shrinkage steel ratio $= \frac{A_s}{b(d+2.5)}$. (When using either p_t curve, mentally redesignate A as A_s.)

CONDITIONS FOR WHICH CHART APPLIES:

1. Sections having moment only, with or without compressive steel.
   $M_s = M$; $A_s = A$
2. Sections having moment and direct force, with or without compressive steel.
   a. Moment and compression force:
   $M_s = M + \frac{Nd''}{12}$; $A_s = A - \frac{N}{20}$,
   the solution does not apply if $A_s < 0$.
   b. Moment and tension force:
   $M_s = M - \frac{Nd''}{12}$; $A_s = A + \frac{N}{20}$

The portion of the chart to the left of the Balanced Stress Line is for singly reinforced sections. The tensile steel is stressed to its allowable limit; the concrete is stressed below its allowable limit.

The portion of the chart to the right of the Balanced Stress Line is for doubly reinforced sections. Both the tensile steel and the concrete are stressed to their allowable limits.

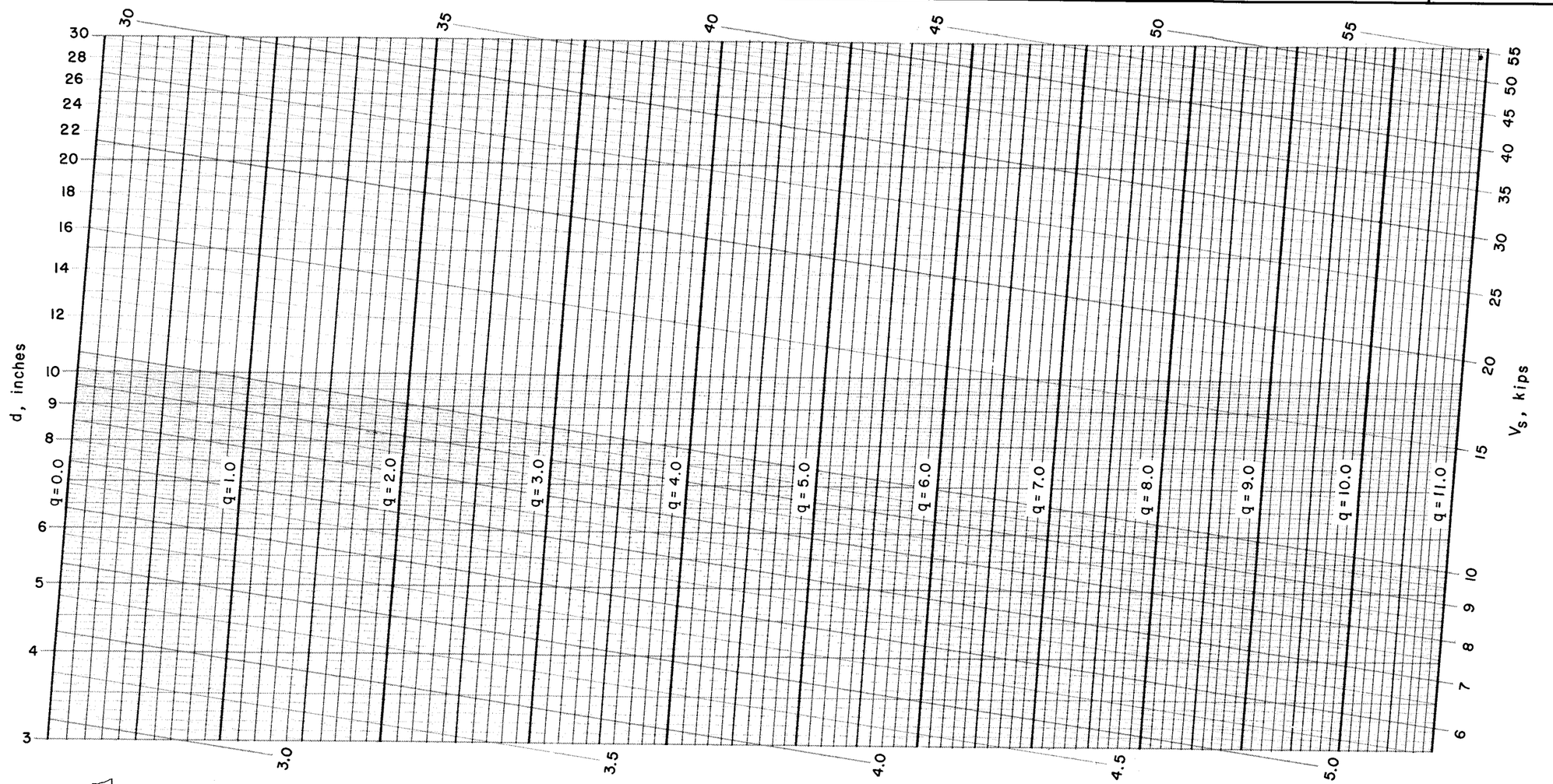
REFERENCE

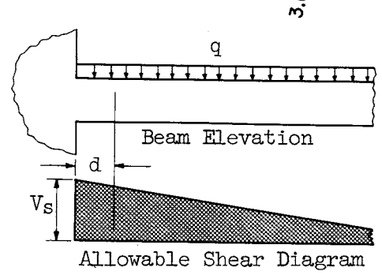
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 166
SHEET 1 OF 3
DATE 4-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
ALLOWABLE TOTAL SHEAR AT FACE OF SUPPORT OF UNIFORMLY LOADED
BEAM WITHOUT WEB REINFORCEMENT. FOR 12 INCH WIDTH.

$f'_c = 5000$ psi





$$V_s = \frac{12 v_c d}{1000}\left(1 + \frac{1000q}{144 v_c}\right)$$

V_s = allowable total shear at face of support, kips
v_c = allowable shear stress at distance equal to [d] from face of support = $1.1\sqrt{f'_c}$, psi
d = effective depth, inches
q = unit load, kips per lineal foot

Solution is valid if, within the interval [d]:
(1) there is no concentrated load, and
(2) the unit load is constant.

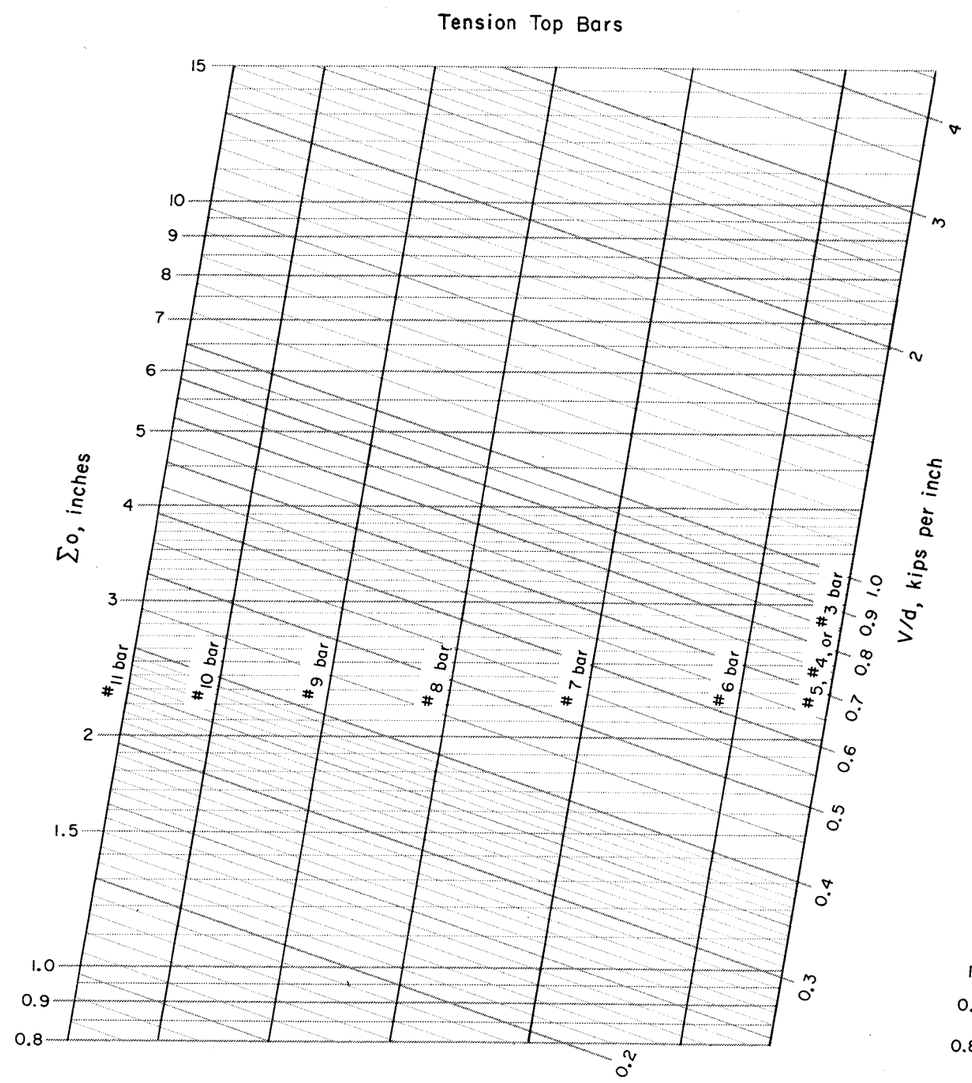
REFERENCE
ACI Building Code (ACI 318-63)

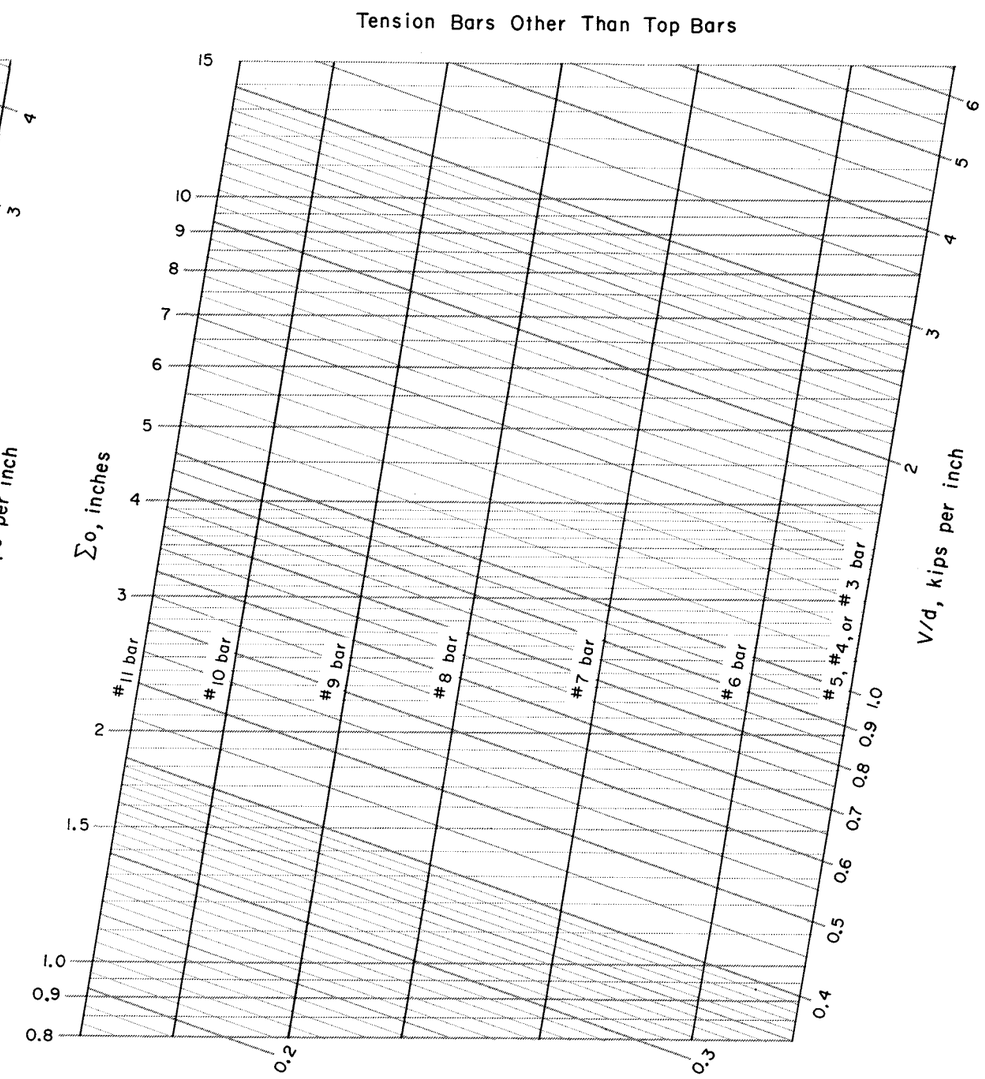
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 166
SHEET 2 OF 3
DATE 7-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
PERIMETER OF TENSILE REINFORCEMENT REQUIRED FOR FLEXURAL BOND.
FOR 12 INCH WIDTH

$f'_c = 5000$ psi





$$\Sigma o = \frac{8000}{7u}(V/d)$$

Σo = required perimeter of tensile reinforcement at the section, inches
V = total shear at the section, kips
d = effective depth, inches

u = allowable bond stress, psi
For tension top bars $u = 3.4\sqrt{f'_c}/D \leq 350$
For tension bars other than top bars $u = 4.8\sqrt{f'_c}/D \leq 500$

REFERENCE

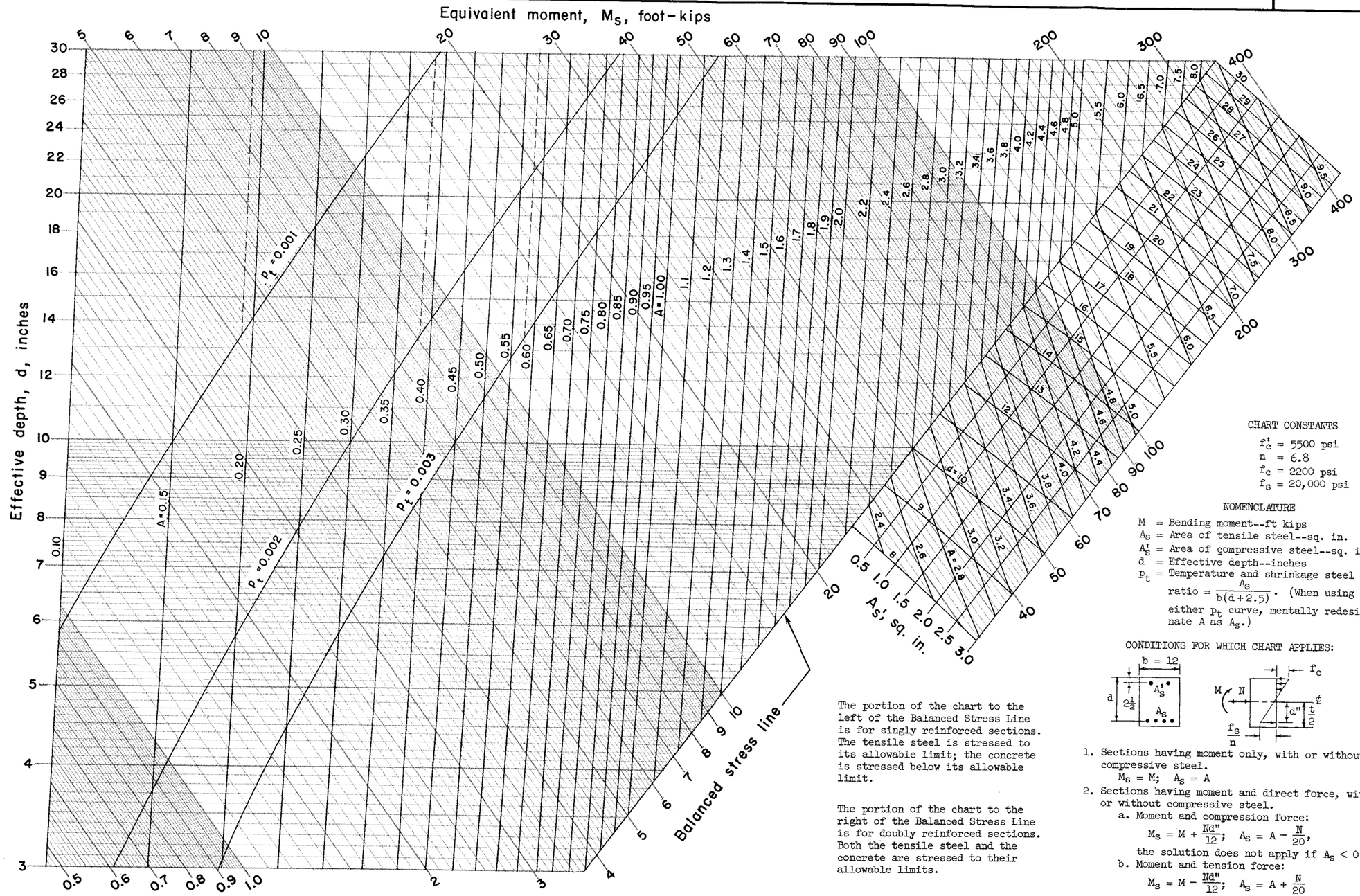
ACI Building Code (ACI 318 - 63)

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 166
SHEET 3 OF 3
DATE 7-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN BENDING ONLY OR BENDING COMBINED WITH DIRECT FORCE. FOR 12 INCH WIDTH

$f'_c = 5500$ psi



The portion of the chart to the left of the Balanced Stress Line is for singly reinforced sections. The tensile steel is stressed to its allowable limit; the concrete is stressed below its allowable limit.

The portion of the chart to the right of the Balanced Stress Line is for doubly reinforced sections. Both the tensile steel and the concrete are stressed to their allowable limits.

CHART CONSTANTS

$f'_c = 5500$ psi
$n = 6.8$
$f_c = 2200$ psi
$f_s = 20{,}000$ psi

NOMENCLATURE

M = Bending moment--ft kips
A_s = Area of tensile steel--sq. in.
A'_s = Area of compressive steel--sq. in.
d = Effective depth--inches
p_t = Temperature and shrinkage steel ratio $= \frac{A_s}{b(d+2.5)}$. (When using either p_t curve, mentally redesignate A as A_s.)

CONDITIONS FOR WHICH CHART APPLIES:

1. Sections having moment only, with or without compressive steel.
   $M_s = M$; $A_s = A$
2. Sections having moment and direct force, with or without compressive steel.
   a. Moment and compression force:
   $M_s = M + \frac{Nd''}{12}$; $A_s = A - \frac{N}{20}$,
   the solution does not apply if $A_s < 0$.
   b. Moment and tension force:
   $M_s = M - \frac{Nd''}{12}$; $A_s = A + \frac{N}{20}$

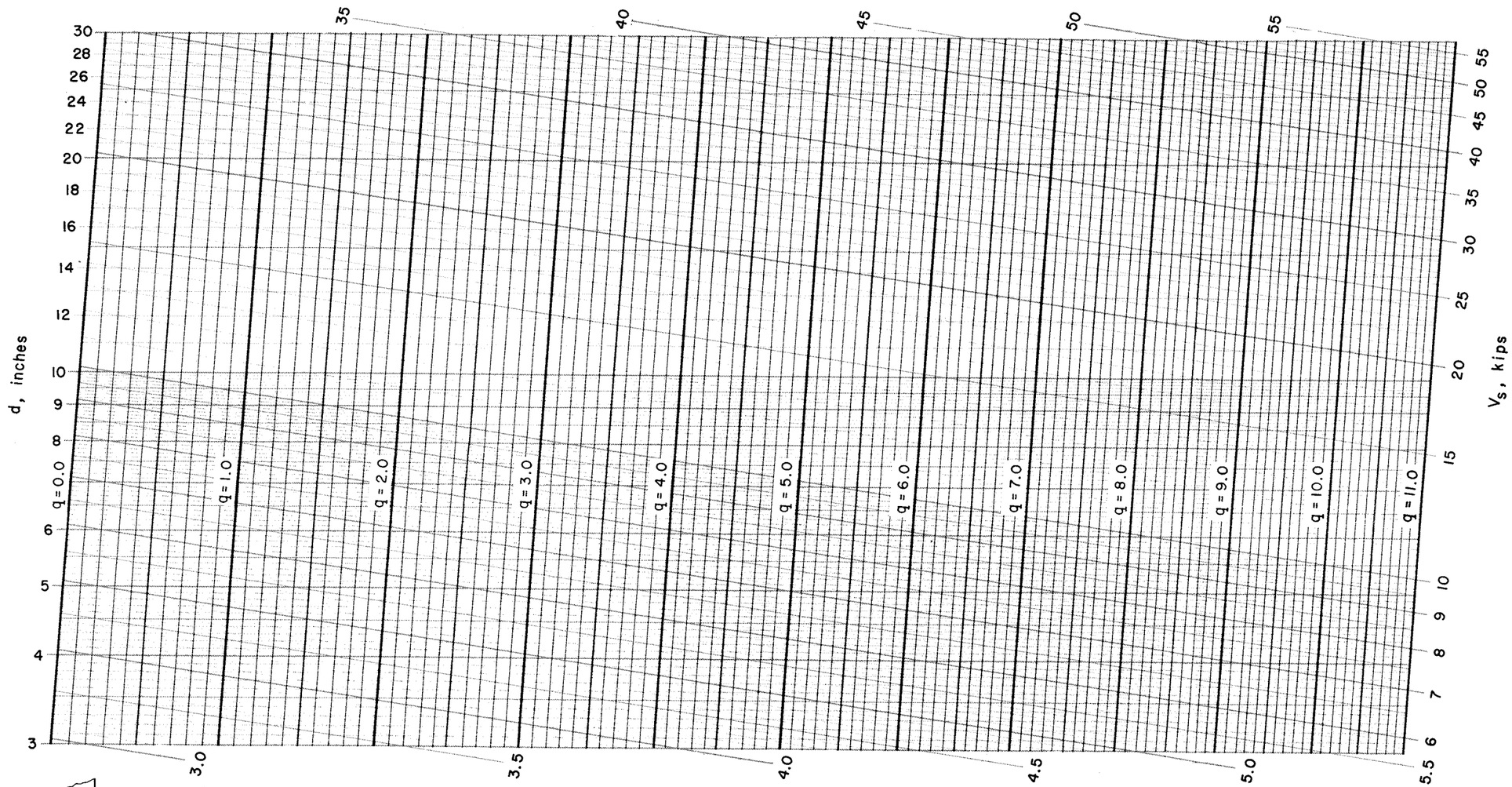
REFERENCE

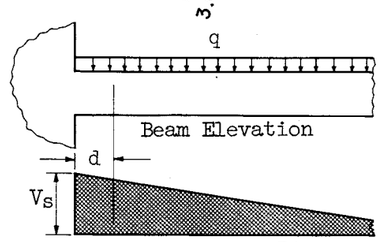
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 167
SHEET 1 OF 3
DATE 4-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
ALLOWABLE TOTAL SHEAR AT FACE OF SUPPORT OF UNIFORMLY LOADED BEAM WITHOUT WEB REINFORCEMENT. FOR 12 INCH WIDTH.

$f'_c = 5500$ psi





Allowable Shear Diagram

$$V_s = \frac{12 v_c d}{1000}\left(1 + \frac{1000q}{144 v_c}\right)$$

V_s = allowable total shear at face of support, kips
v_c = allowable shear stress at distance equal to [d] from face of support = $1.1\sqrt{f'_c}$, psi
d = effective depth, inches
q = unit load, kips per lineal foot

Solution is valid if, within the interval [d]:
(1) there is no concentrated load, and
(2) the unit load is constant.

REFERENCE
ACI Building Code (ACI 318-63)

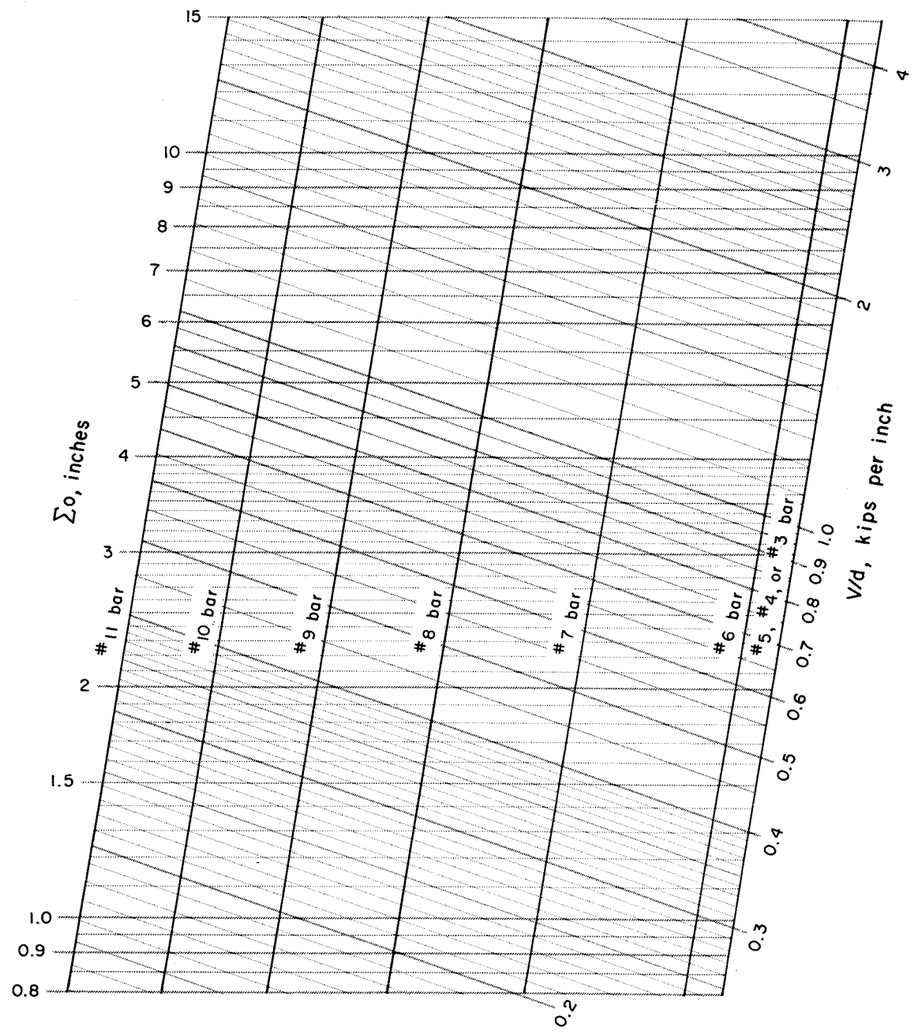
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 167
SHEET 2 OF 3
DATE 7-64

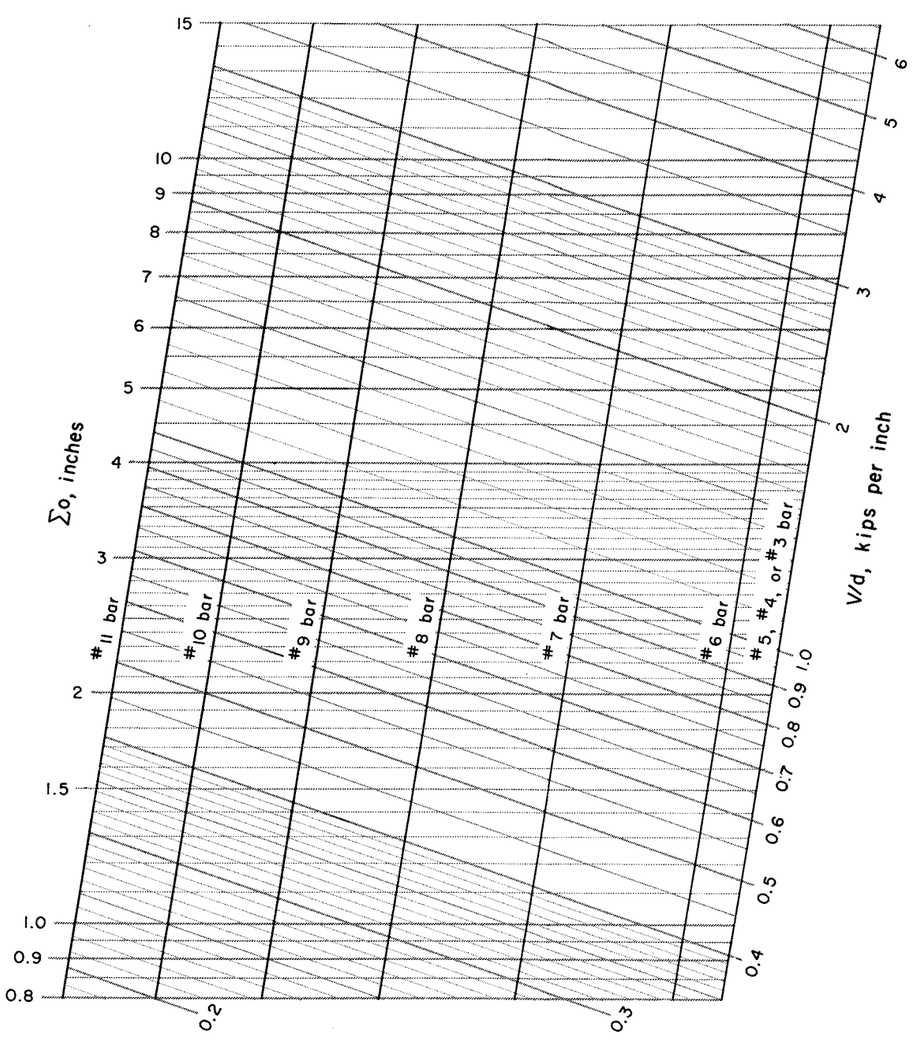
STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
PERIMETER OF TENSILE REINFORCEMENT REQUIRED FOR FLEXURAL BOND.
FOR 12 INCH WIDTH

$f'_c = 5500$ psi

Tension Top Bars



Tension Bars Other Than Top Bars



$$\Sigma o = \frac{8000}{7u}(V/d)$$

Σo = required perimeter of tensile reinforcement at the section, inches
V = total shear at the section, kips
d = effective depth, inches

u = allowable bond stress, psi
For tension top bars $u = 3.4\sqrt{f'_c}/D \leq 350$
For tension bars other than top bars $u = 4.8\sqrt{f'_c}/D \leq 500$

REFERENCE
ACI Building Code (ACI 318-63)

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES-167
SHEET 3 OF 3
DATE 7-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN BENDING ONLY OR BENDING COMBINED WITH DIRECT FORCE. FOR 12 INCH WIDTH

$f'_c = 6000$ psi

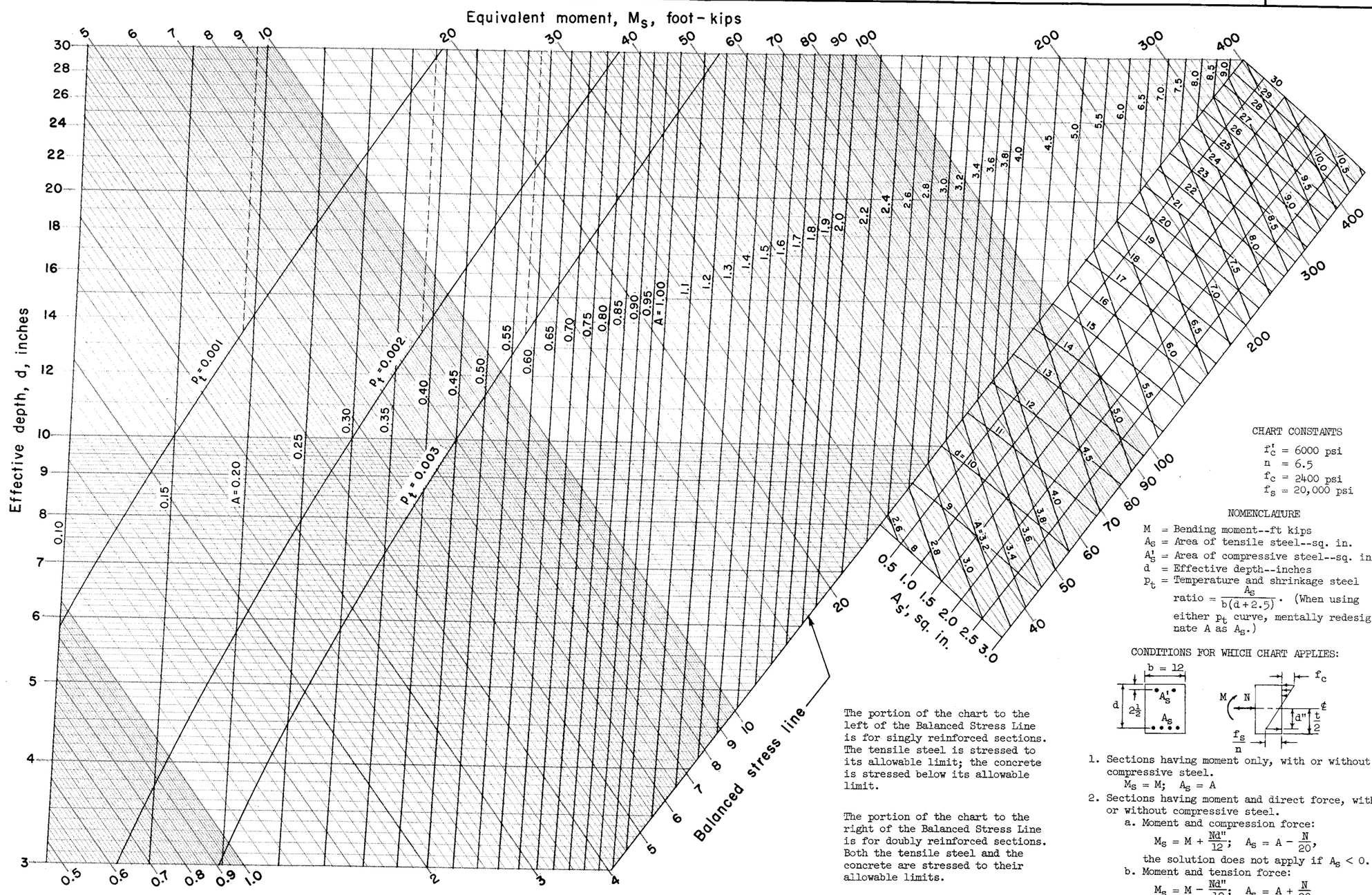


CHART CONSTANTS

f'_c = 6000 psi
n = 6.5
f_c = 2400 psi
f_s = 20,000 psi

NOMENCLATURE

M = Bending moment--ft kips
A_s = Area of tensile steel--sq. in.
A'_s = Area of compressive steel--sq. in.
d = Effective depth--inches
p_t = Temperature and shrinkage steel ratio = $\frac{A_s}{b(d+2.5)}$. (When using either p_t curve, mentally redesignate A as A_s.)

CONDITIONS FOR WHICH CHART APPLIES:

1. Sections having moment only, with or without compressive steel.
   $M_s = M$; $A_s = A$
2. Sections having moment and direct force, with or without compressive steel.
   a. Moment and compression force:
   $M_s = M + \frac{Nd''}{12}$; $A_s = A - \frac{N}{20}$,
   the solution does not apply if $A_s < 0$.
   b. Moment and tension force:
   $M_s = M - \frac{Nd''}{12}$; $A_s = A + \frac{N}{20}$

The portion of the chart to the left of the Balanced Stress Line is for singly reinforced sections. The tensile steel is stressed to its allowable limit; the concrete is stressed below its allowable limit.

The portion of the chart to the right of the Balanced Stress Line is for doubly reinforced sections. Both the tensile steel and the concrete are stressed to their allowable limits.

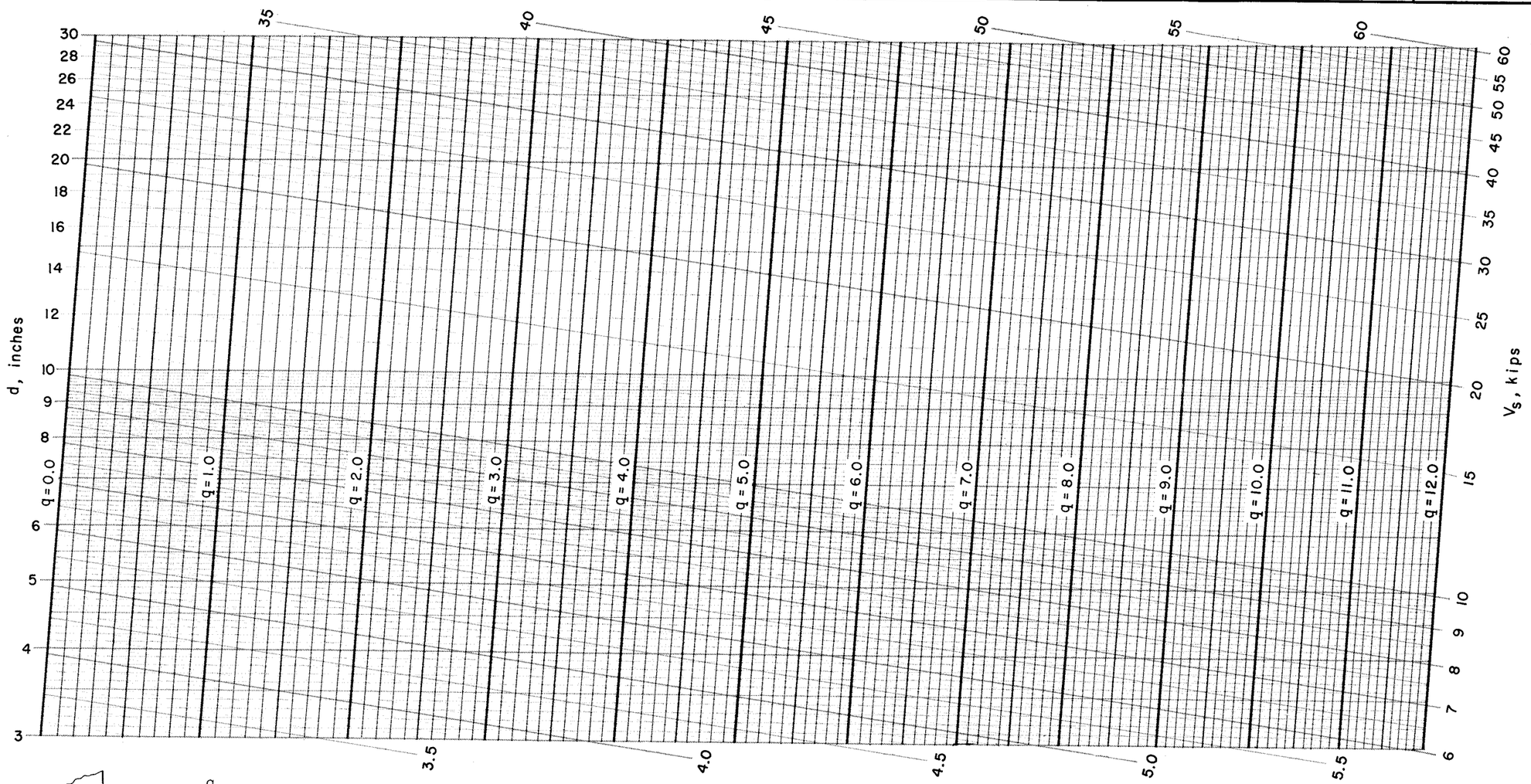
REFERENCE

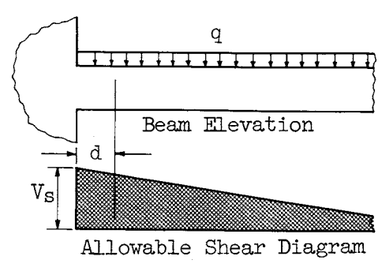
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO. ES- 168
SHEET 1 OF 3
DATE 4-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
ALLOWABLE TOTAL SHEAR AT FACE OF SUPPORT OF UNIFORMLY LOADED BEAM WITHOUT WEB REINFORCEMENT. FOR 12 INCH WIDTH.

$f'_c = 6000$ psi





$$V_s = \frac{12 v_c d}{1000}\left(1 + \frac{1000q}{144 v_c}\right)$$

V_s = allowable total shear at face of support, kips
v_c = allowable shear stress at distance equal to [d] from face of support = $1.1\sqrt{f'_c}$, psi
d = effective depth, inches
q = unit load, kips per lineal foot

Solution is valid if, within the interval [d]:
(1) there is no concentrated load, and
(2) the unit load is constant.

REFERENCE
ACI Building Code (ACI 318-63)

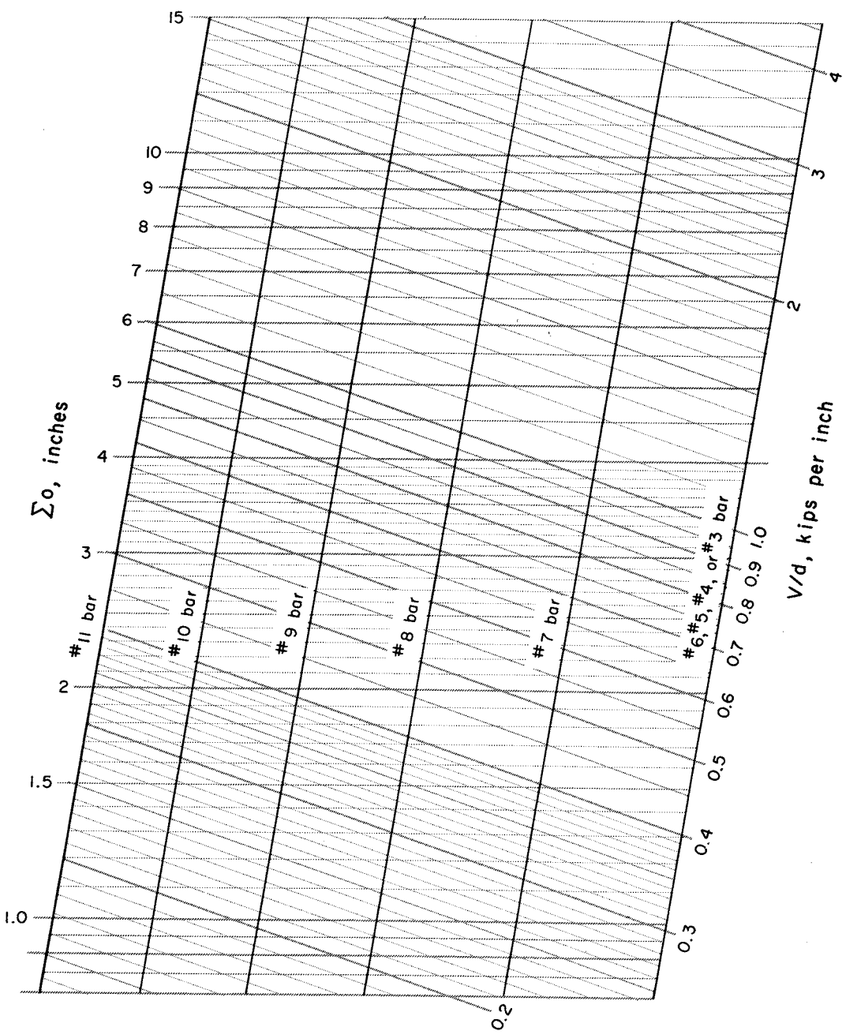
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 168
SHEET 2 OF 3
DATE 7-64

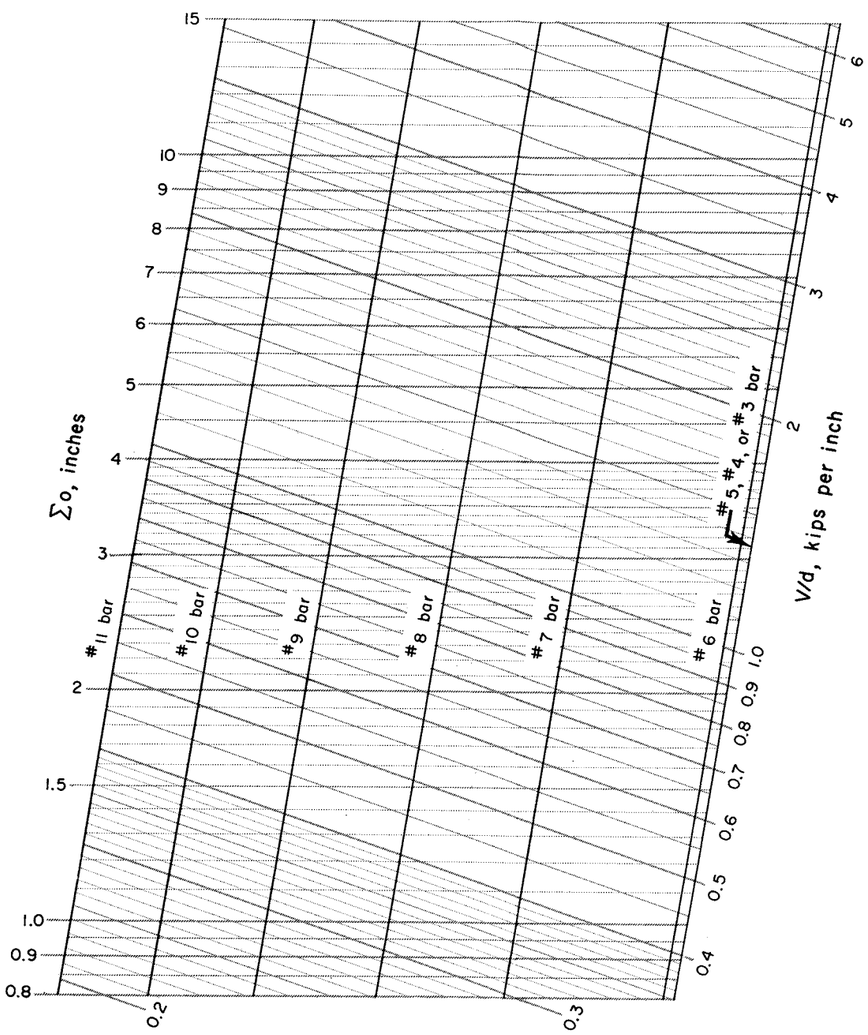
STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
PERIMETER OF TENSILE REINFORCEMENT REQUIRED FOR FLEXURAL BOND.
FOR 12 INCH WIDTH

$f'_c = 6000$ psi

Tension Top Bars



Tension Bars Other Than Top Bars



$$\Sigma o = \frac{8000}{7u}(V/d)$$

Σo = required perimeter of tensile reinforcement at the section, inches
V = total shear at the section, kips
d = effective depth, inches

u = allowable bond stress, psi
For tension top bars $u = 3.4\sqrt{f'_c}/D \leq 350$
For tension bars other than top bars $u = 4.8\sqrt{f'_c}/D \leq 500$

REFERENCE

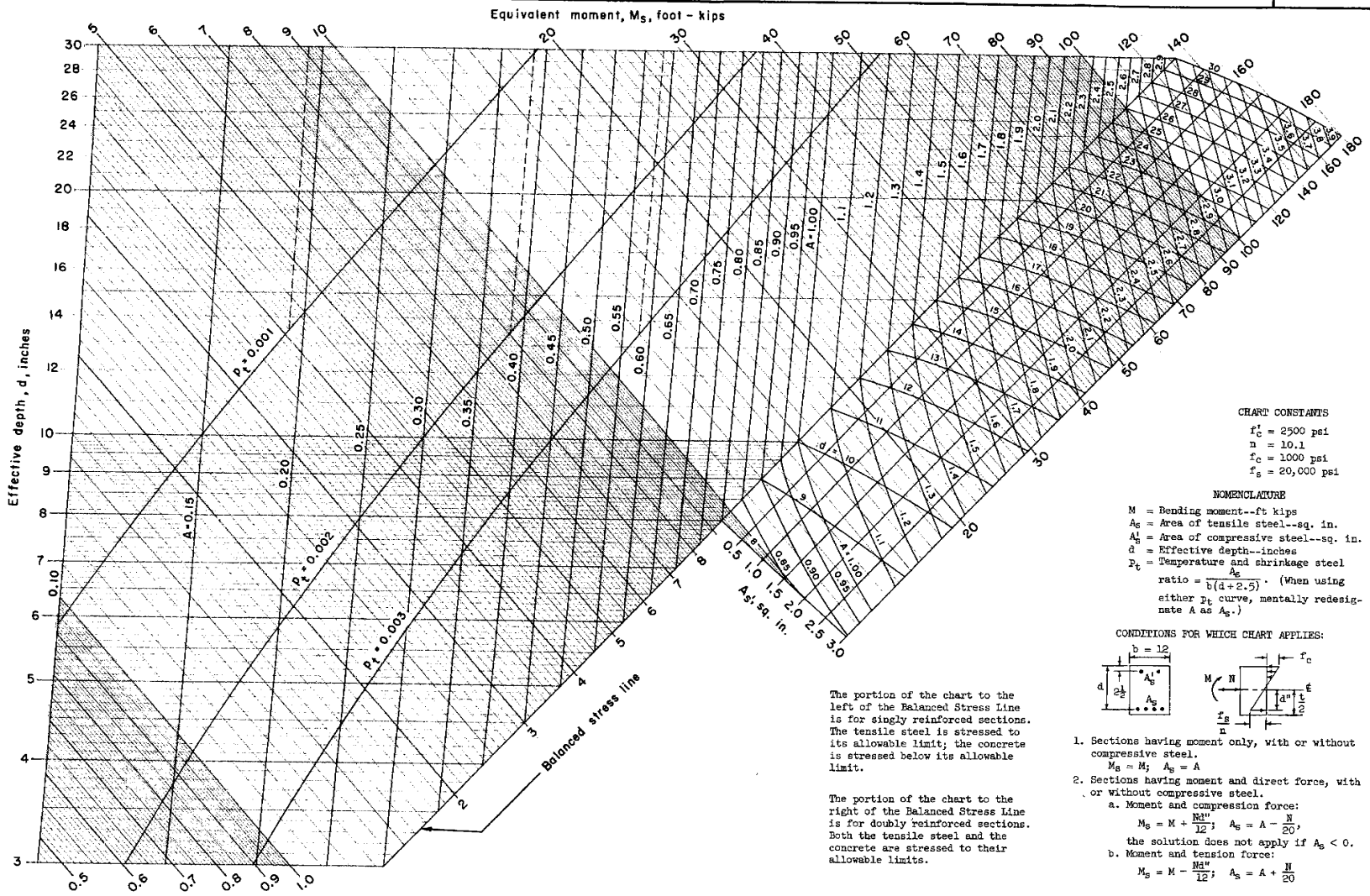
ACI Building Code (ACI 318-63)

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 168
SHEET 3 OF 3
DATE 7-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN BENDING ONLY OR BENDING COMBINED WITH DIRECT FORCE. FOR 12 INCH WIDTH

$f'_c = 2500$ psi



The portion of the chart to the left of the Balanced Stress Line is for singly reinforced sections. The tensile steel is stressed to its allowable limit; the concrete is stressed below its allowable limit.

The portion of the chart to the right of the Balanced Stress Line is for doubly reinforced sections. Both the tensile steel and the concrete are stressed to their allowable limits.

CHART CONSTANTS

$f'_c = 2500$ psi
$n = 10.1$
$f_c = 1000$ psi
$f_s = 20{,}000$ psi

NOMENCLATURE

M = Bending moment--ft kips
A_s = Area of tensile steel--sq. in.
A'_s = Area of compressive steel--sq. in.
d = Effective depth--inches
p_t = Temperature and shrinkage steel ratio $= \frac{A_s}{b(d+2.5)}$. (When using either p_t curve, mentally redesignate A as A_s.)

CONDITIONS FOR WHICH CHART APPLIES:

1. Sections having moment only, with or without compressive steel.
   $M_s = M$; $A_s = A$
2. Sections having moment and direct force, with or without compressive steel.
   a. Moment and compression force:
   $M_s = M + \frac{Nd''}{12}$; $A_s = A - \frac{N}{20}$,
   the solution does not apply if $A_s < 0$.
   b. Moment and tension force:
   $M_s = M - \frac{Nd''}{12}$; $A_s = A + \frac{N}{20}$

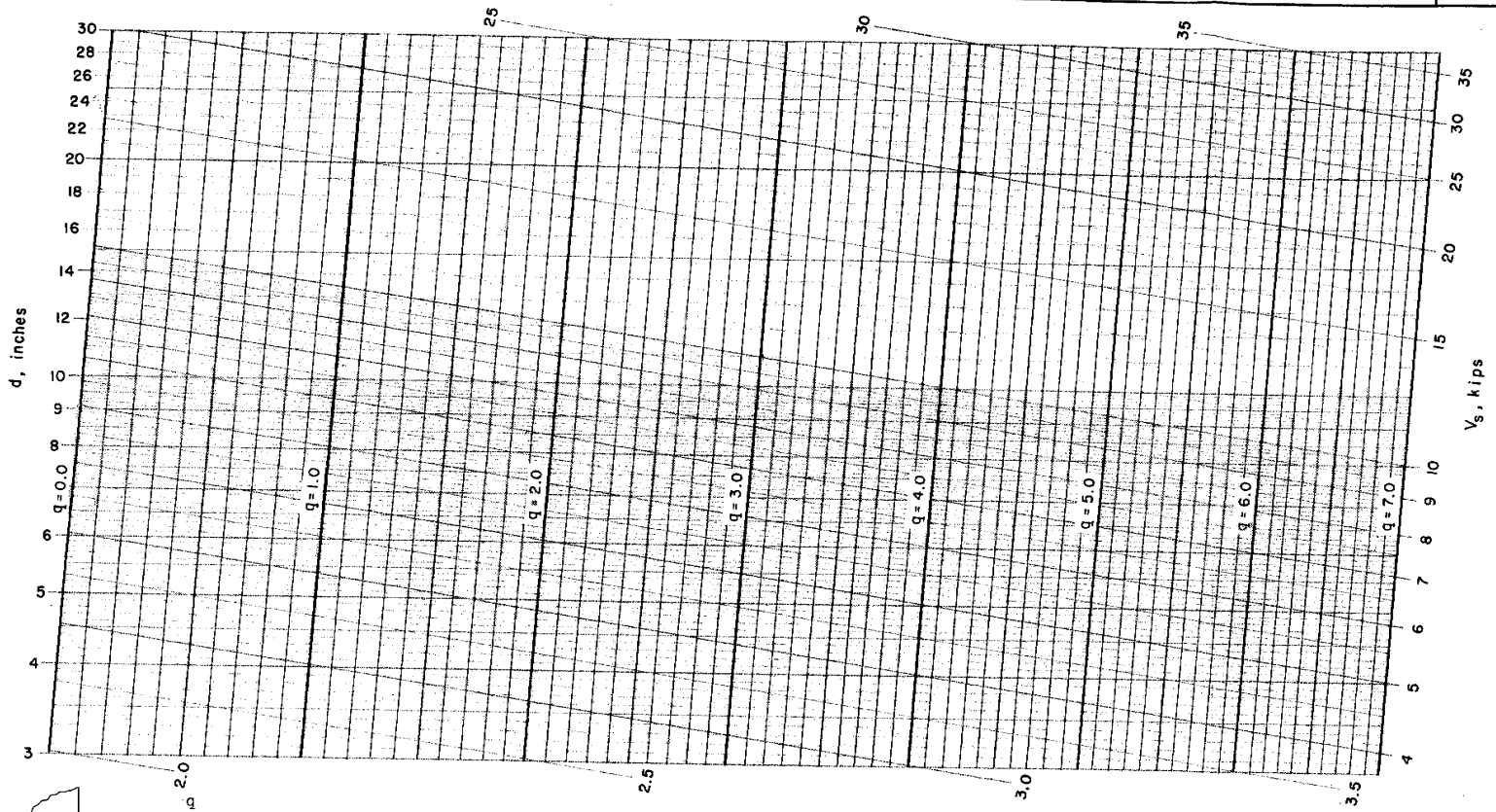
REFERENCE

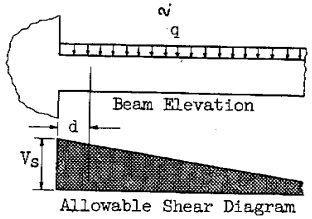
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION · DESIGN SECTION

STANDARD DWG. NO. ES- 161
SHEET 1 OF 3
DATE 4-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
ALLOWABLE TOTAL SHEAR AT FACE OF SUPPORT OF UNIFORMLY LOADED BEAM WITHOUT WEB REINFORCEMENT. FOR 12 INCH WIDTH.

$f'_c = 2500$ psi





$$V_s = \frac{12 v_c d}{1000}\left(1. + \frac{1000q}{144 v_c}\right)$$

V_s = allowable total shear at face of support, kips
v_c = allowable shear stress at distance equal to [d] from face of support = $1.1\sqrt{f'_c}$, psi
d = effective depth, inches
q = unit load, kips per lineal foot

Solution is valid if, within the interval [d]:
(1) there is no concentrated load, and
(2) the unit load is constant.

REFERENCE
ACI Building Code (ACI 318-63)

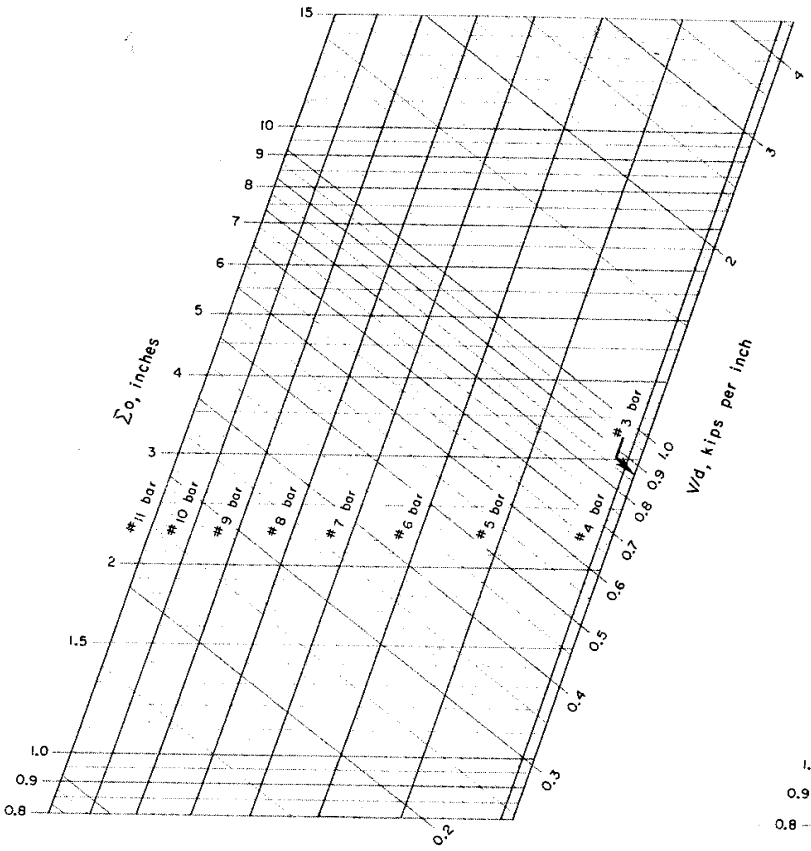
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO. ES-161
SHEET 2 OF 3
DATE 7-64

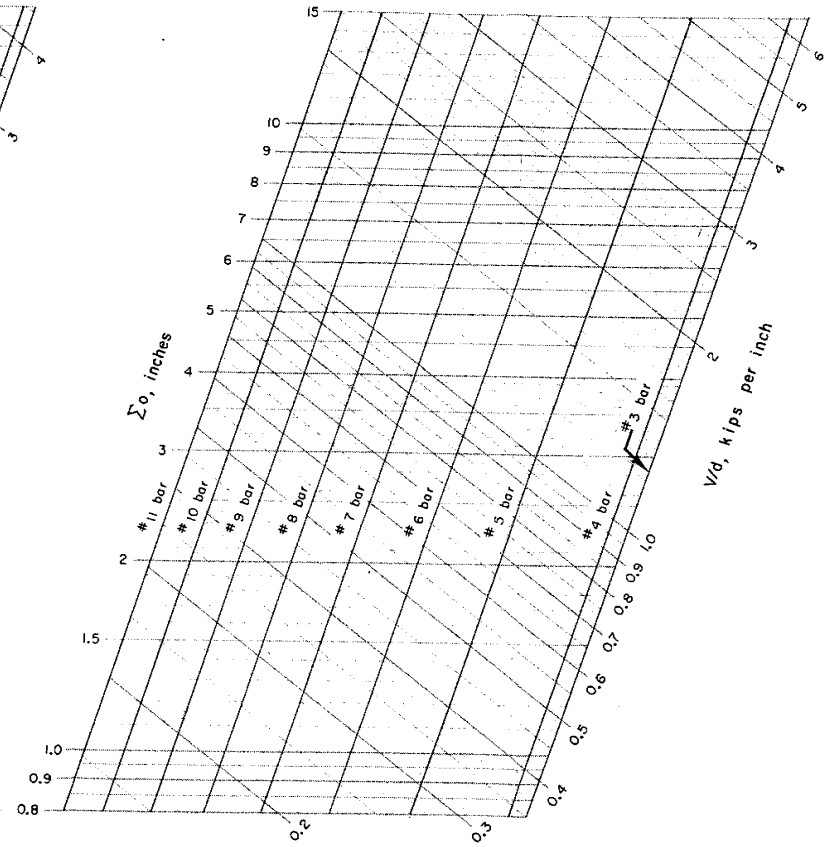
STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
PERIMETER OF TENSILE REINFORCEMENT REQUIRED FOR FLEXURAL BOND.
FOR 12 INCH WIDTH

$f'_c = 2500$ psi

Tension Top Bars



Tension Bars Other Than Top Bars



$$\Sigma o = \frac{8000}{7u}(V/d)$$

Σo = required perimeter of tensile reinforcement at the section, inches
V = total shear at the section, kips
d = effective depth, inches

u = allowable bond stress, psi
For tension top bars $u = 3.4\sqrt{f'_c}/D \leq 350$
For tension bars other than top bars $u = 4.8\sqrt{f'_c}/D \leq 500$

REFERENCE

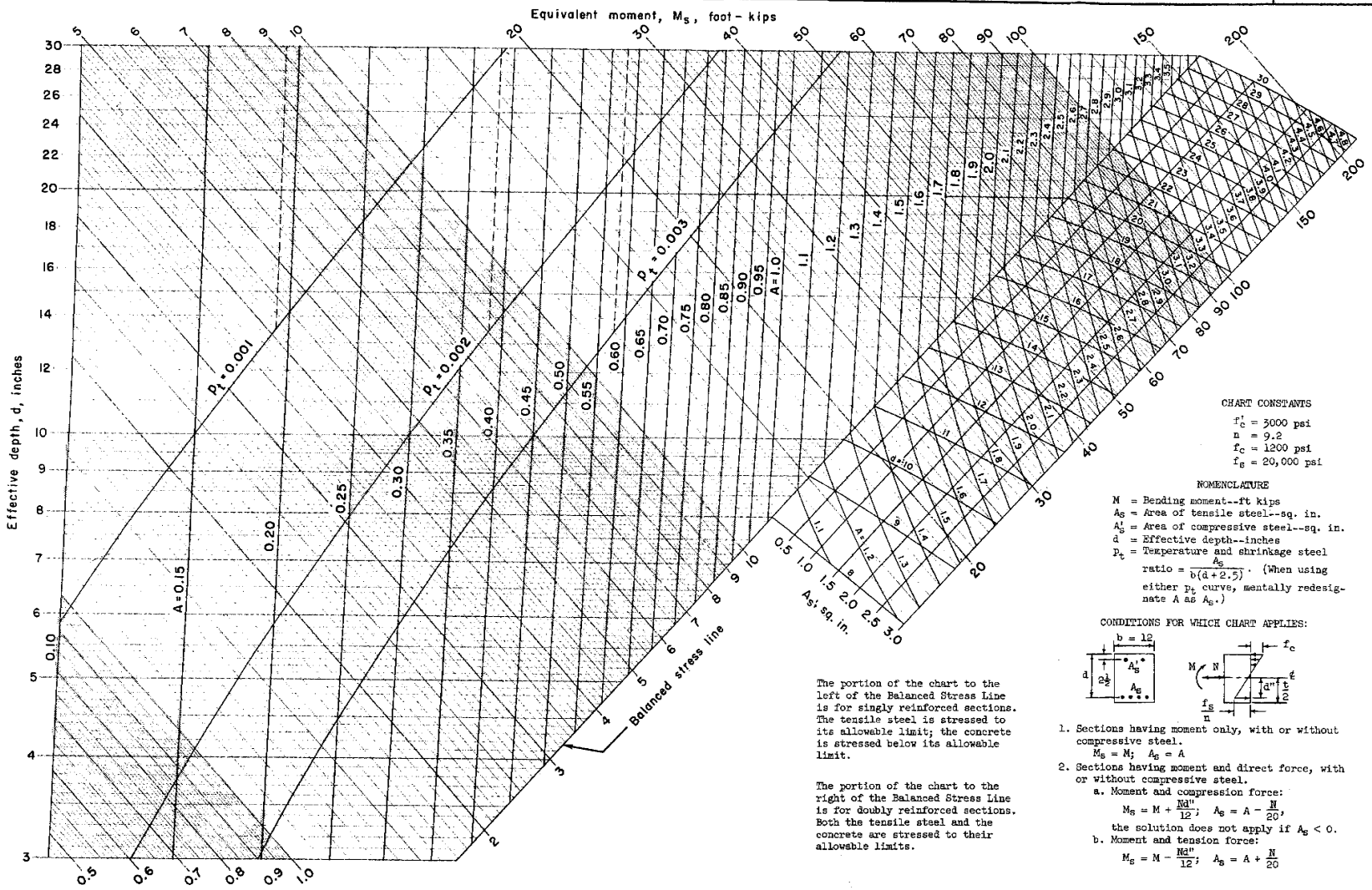
ACI Building Code (ACI 318-63)

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES-161
SHEET 3 OF 3
DATE 7-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN BENDING ONLY OR BENDING COMBINED WITH DIRECT FORCE. FOR 12 INCH WIDTH

$f'_c = 3000$ psi



The portion of the chart to the left of the Balanced Stress Line is for singly reinforced sections. The tensile steel is stressed to its allowable limit; the concrete is stressed below its allowable limit.

The portion of the chart to the right of the Balanced Stress Line is for doubly reinforced sections. Both the tensile steel and the concrete are stressed to their allowable limits.

CHART CONSTANTS

$f'_c = 3000$ psi
$n = 9.2$
$f_c = 1200$ psi
$f_s = 20{,}000$ psi

NOMENCLATURE

M = Bending moment--ft kips
A_s = Area of tensile steel--sq. in.
A'_s = Area of compressive steel--sq. in.
d = Effective depth--inches
p_t = Temperature and shrinkage steel ratio $= \frac{A_s}{b(d+2.5)}$. (When using either p_t curve, mentally redesignate A as A_s.)

CONDITIONS FOR WHICH CHART APPLIES:

1. Sections having moment only, with or without compressive steel.
   $M_s = M$; $A_s = A$
2. Sections having moment and direct force, with or without compressive steel.
   a. Moment and compression force:
   $M_s = M + \frac{Nd''}{12}$; $A_s = A - \frac{N}{20}$,
   the solution does not apply if $A_s < 0$.
   b. Moment and tension force:
   $M_s = M - \frac{Nd''}{12}$; $A_s = A + \frac{N}{20}$

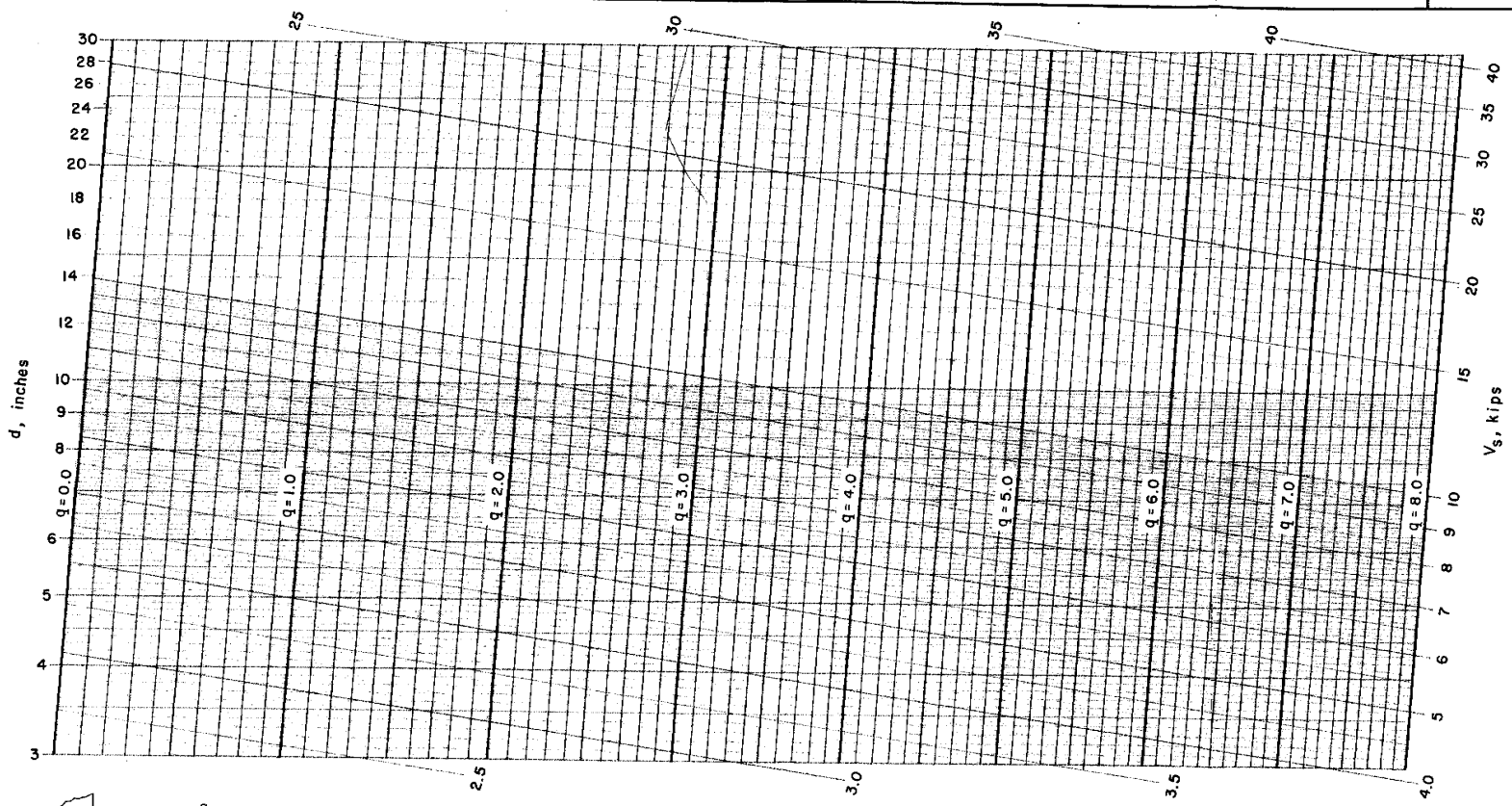
REFERENCE

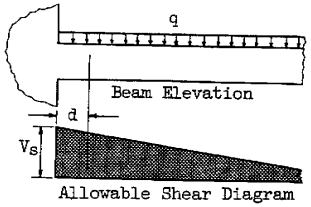
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 162
SHEET 1 OF 3
DATE 4-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
ALLOWABLE TOTAL SHEAR AT FACE OF SUPPORT OF UNIFORMLY LOADED BEAM WITHOUT WEB REINFORCEMENT. FOR 12 INCH WIDTH.

$f'_c = 3000$ psi





$$V_s = \frac{12 v_c d}{1000}\left(1 + \frac{1000q}{144 v_c}\right)$$

V_s = allowable total shear at face of support, kips
v_c = allowable shear stress at distance equal to [d] from face of support = $1.1\sqrt{f'_c}$, psi
d = effective depth, inches
q = unit load, kips per lineal foot

Solution is valid if, within the interval [d]:
(1) there is no concentrated load, and
(2) the unit load is constant.

REFERENCE
ACI Building Code (ACI 318-63)

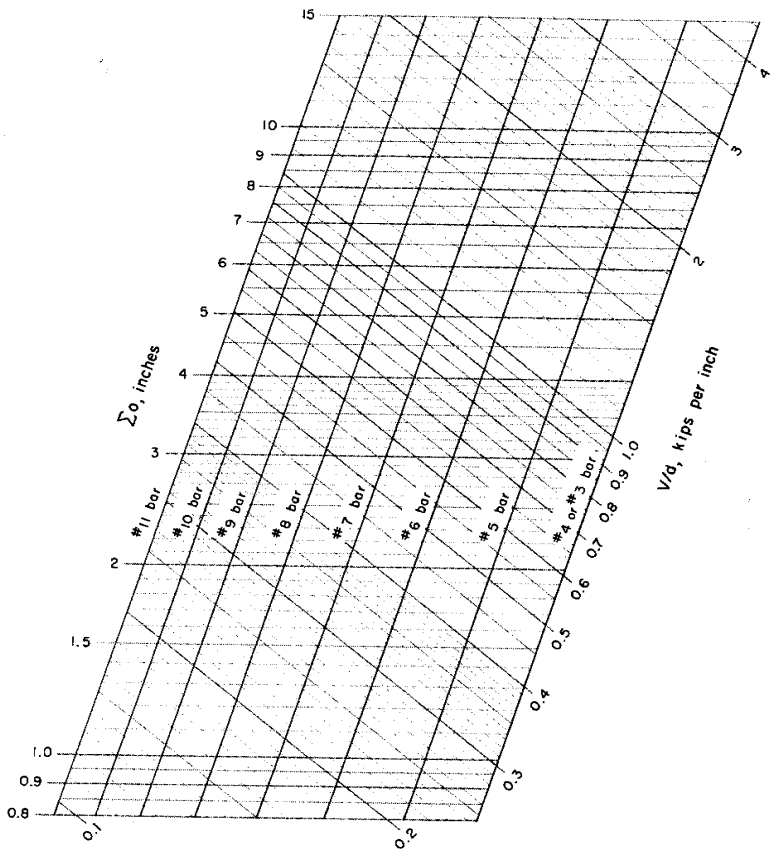
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 162
SHEET 2 OF 3
DATE 7-64

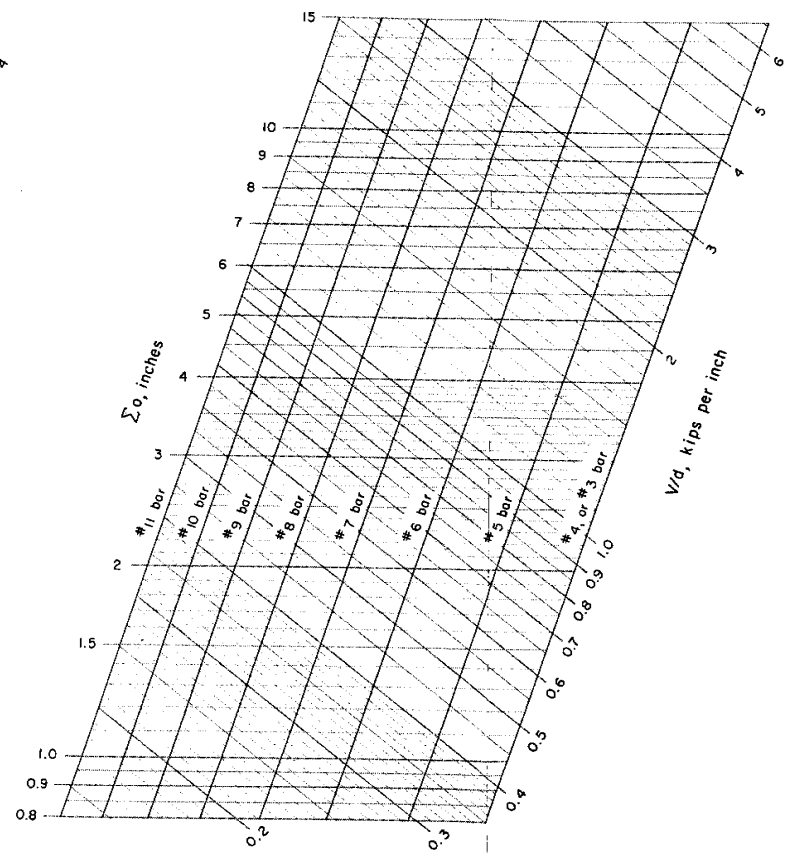
STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
PERIMETER OF TENSILE REINFORCEMENT REQUIRED FOR FLEXURAL BOND.
FOR 12 INCH WIDTH

$f'_c = 3000$ psi

Tension Top Bars



Tension Bars Other Than Top Bars



$$\Sigma o = \frac{8000}{7u}(V/d)$$

Σo = required perimeter of tensile reinforcement at the section, inches
V = total shear at the section, kips
d = effective depth, inches

u = allowable bond stress, psi
For tension top bars $u = 3.4\sqrt{f'_c}/D \leq 350$
For tension bars other than top bars $u = 4.8\sqrt{f'_c}/D \leq 500$

REFERENCE

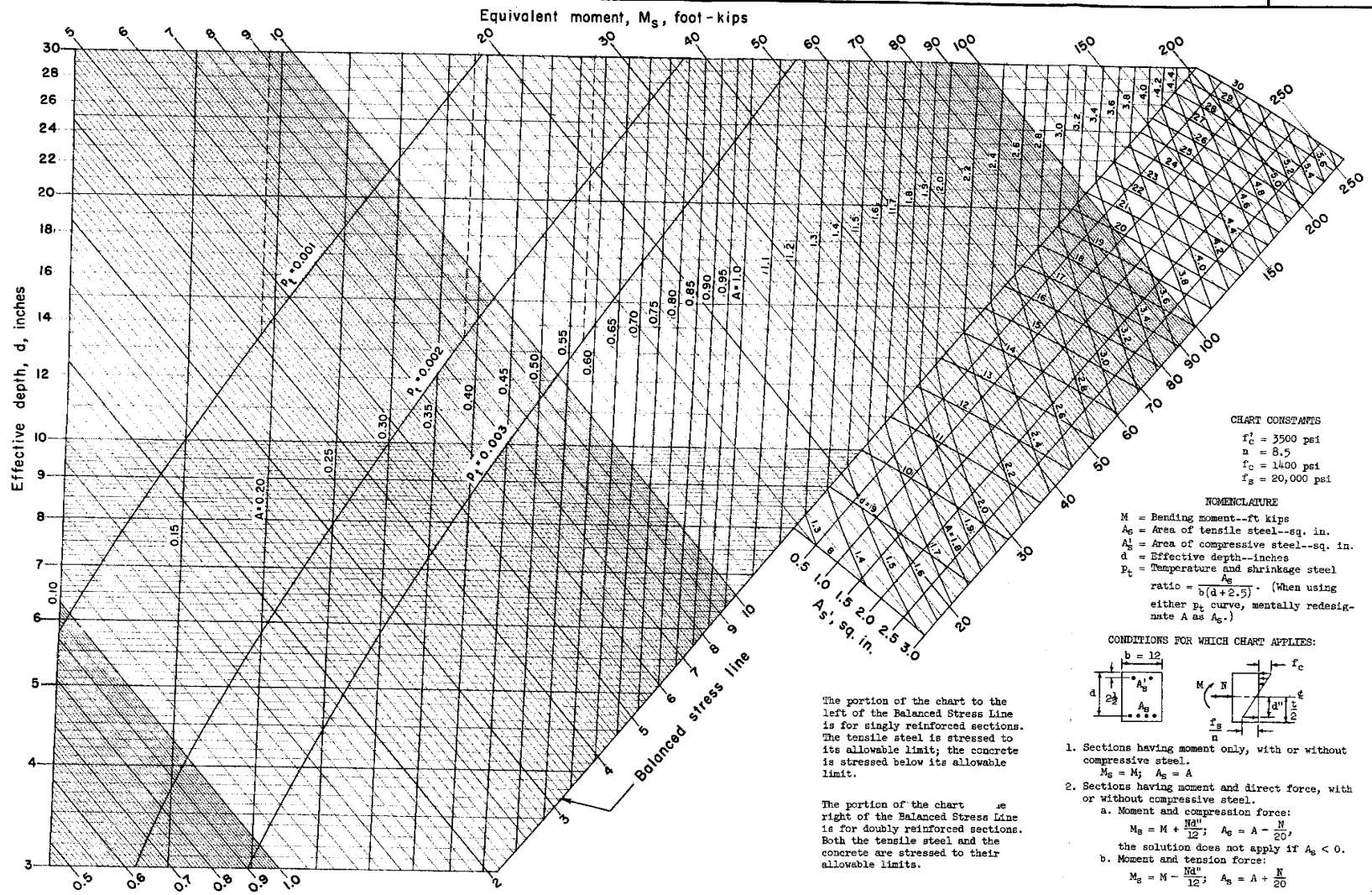
ACI Building Code (ACI 318 - 63)

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES-162
SHEET 3 OF 3
DATE 7-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN BENDING ONLY OR BENDING COMBINED WITH DIRECT FORCE. FOR 12 INCH WIDTH

$f'_c = 3500$ psi



The portion of the chart to the left of the Balanced Stress Line is for singly reinforced sections. The tensile steel is stressed to its allowable limit; the concrete is stressed below its allowable limit.

The portion of the chart [illegible]e right of the Balanced Stress Line is for doubly reinforced sections. Both the tensile steel and the concrete are stressed to their allowable limits.

CHART CONSTANTS

$f'_c = 3500$ psi
$n = 8.5$
$f_c = 1400$ psi
$f_s = 20{,}000$ psi

NOMENCLATURE

M = Bending moment--ft kips
A_s = Area of tensile steel--sq. in.
A'_s = Area of compressive steel--sq. in.
d = Effective depth--inches
p_t = Temperature and shrinkage steel ratio = $\frac{A_s}{b(d+2.5)}$. (When using either p_t curve, mentally redesignate A as A_s.)

CONDITIONS FOR WHICH CHART APPLIES:

1. Sections having moment only, with or without compressive steel.
   $M_s = M$; $A_s = A$
2. Sections having moment and direct force, with or without compressive steel.
   a. Moment and compression force:
   $M_s = M + \frac{Nd''}{12}$; $A_s = A - \frac{N}{20}$,
   the solution does not apply if $A_s < 0$.
   b. Moment and tension force:
   $M_s = M - \frac{Nd''}{12}$; $A_s = A + \frac{N}{20}$

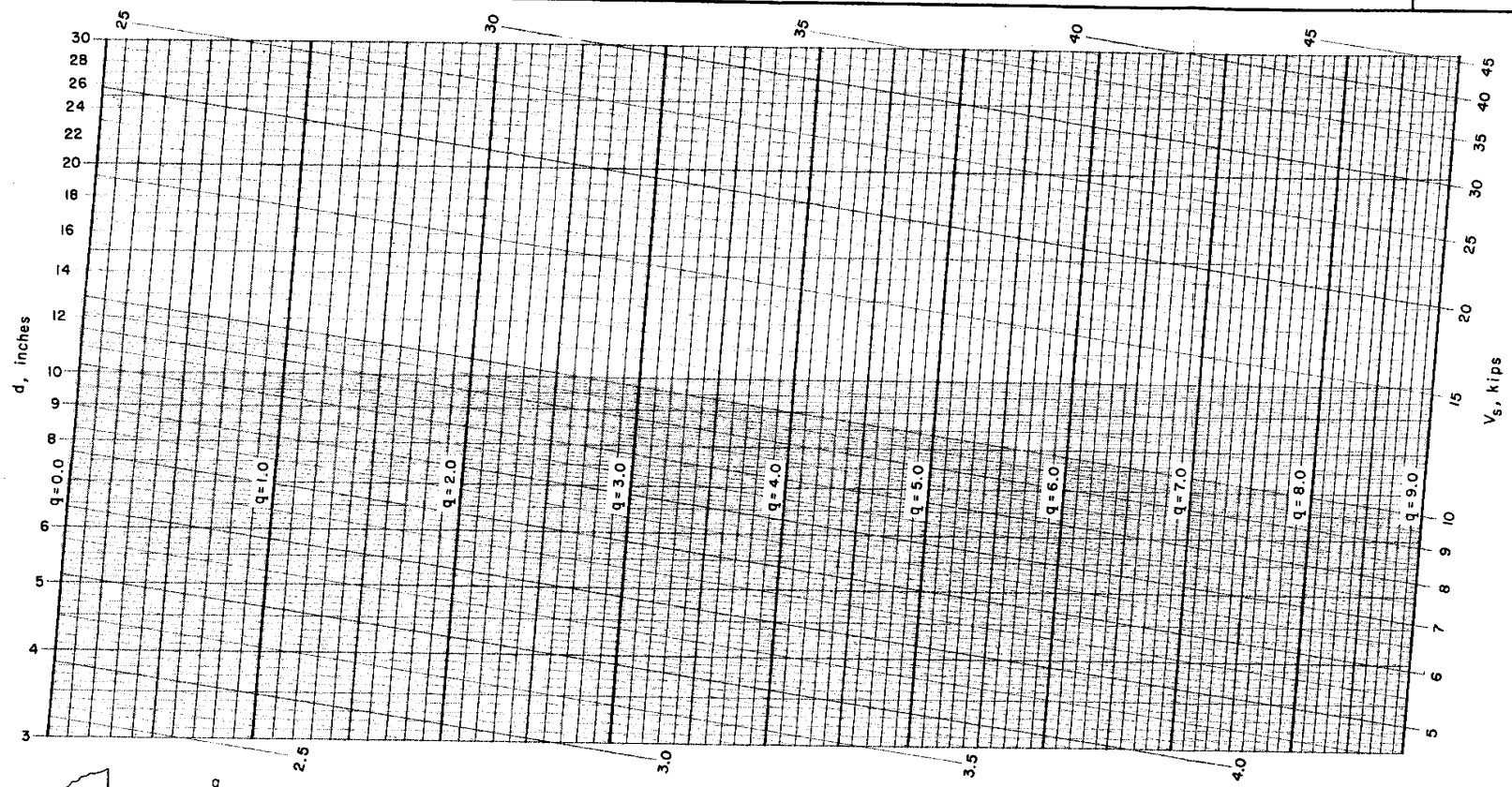
REFERENCE

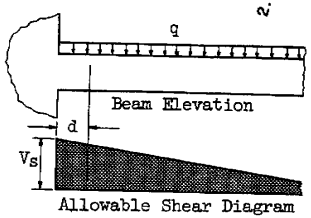
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION, DESIGN SECTION

STANDARD DWG. NO. ES- 163
SHEET 1 OF 3
DATE 4-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
ALLOWABLE TOTAL SHEAR AT FACE OF SUPPORT OF UNIFORMLY LOADED
BEAM WITHOUT WEB REINFORCEMENT. FOR 12 INCH WIDTH.

$f'_c = 3500$ psi





$$V_s = \frac{12 v_c d}{1000}\left(1 + \frac{1000q}{144 v_c}\right)$$

V_s = allowable total shear at face of support, kips
v_c = allowable shear stress at distance equal to [d] from face of support = $1.1\sqrt{f'_c}$, psi
d = effective depth, inches
q = unit load, kips per lineal foot

Solution is valid if, within the interval [d]:
(1) there is no concentrated load, and
(2) the unit load is constant.

REFERENCE
ACI Building Code (ACI 318-63)

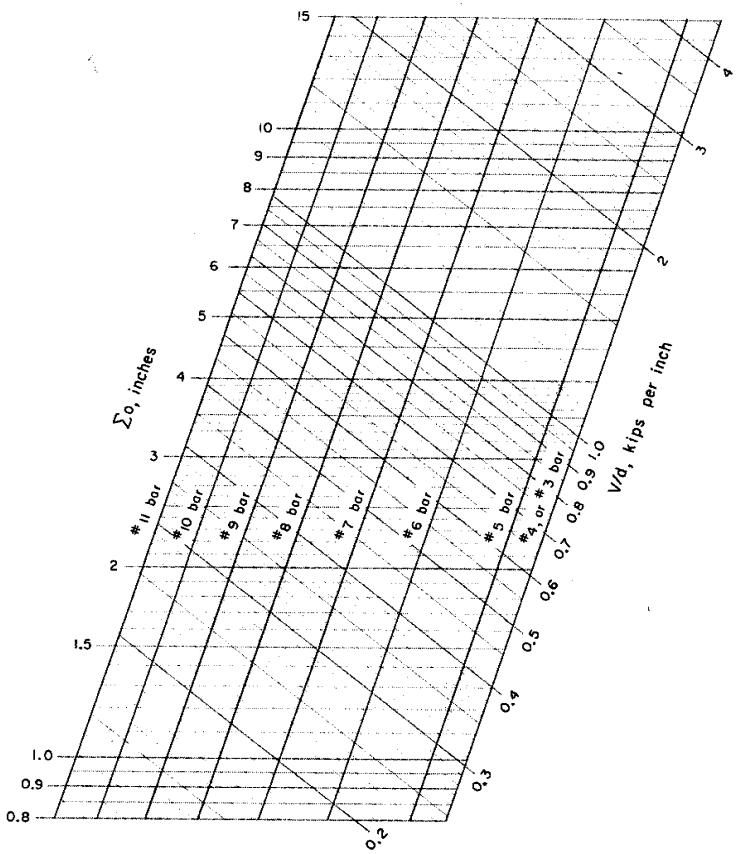
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 163
SHEET 2 OF 3
DATE 7-64

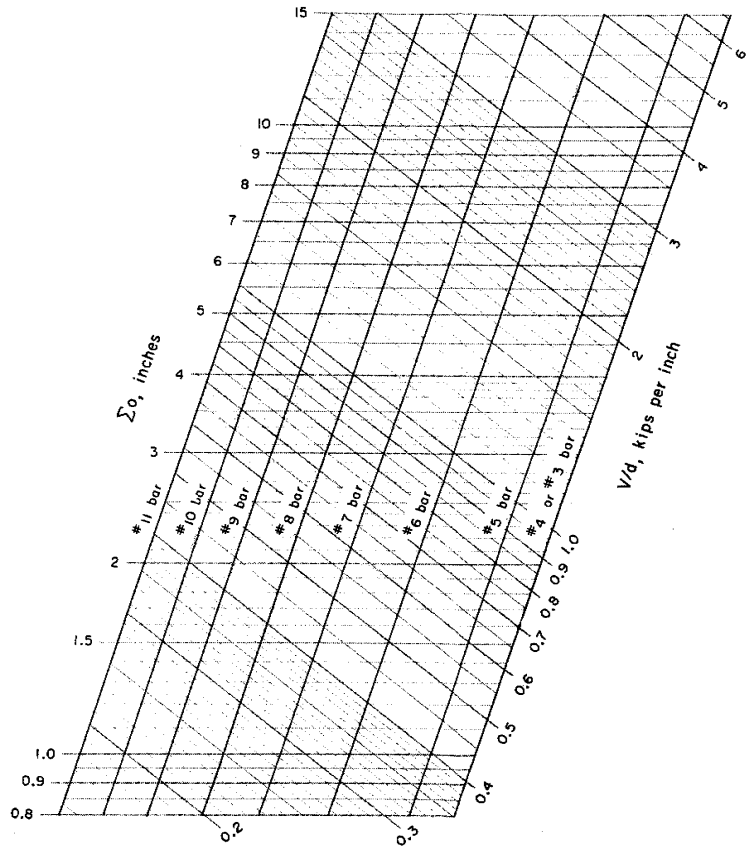
STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
PERIMETER OF TENSILE REINFORCEMENT REQUIRED FOR FLEXURAL BOND.
FOR 12 INCH WIDTH

$f'_c = 3500$ psi

Tension Top Bars



Tension Bars Other Than Top Bars



$$\Sigma o = \frac{8000}{7u}(V/d)$$

Σo = required perimeter of tensile reinforcement at the section, inches
V = total shear at the section, kips
d = effective depth, inches

u = allowable bond stress, psi
For tension top bars $u = 3.4\sqrt{f'_c}/D \leq 350$
For tension bars other than top bars $u = 4.8\sqrt{f'_c}/D \leq 500$

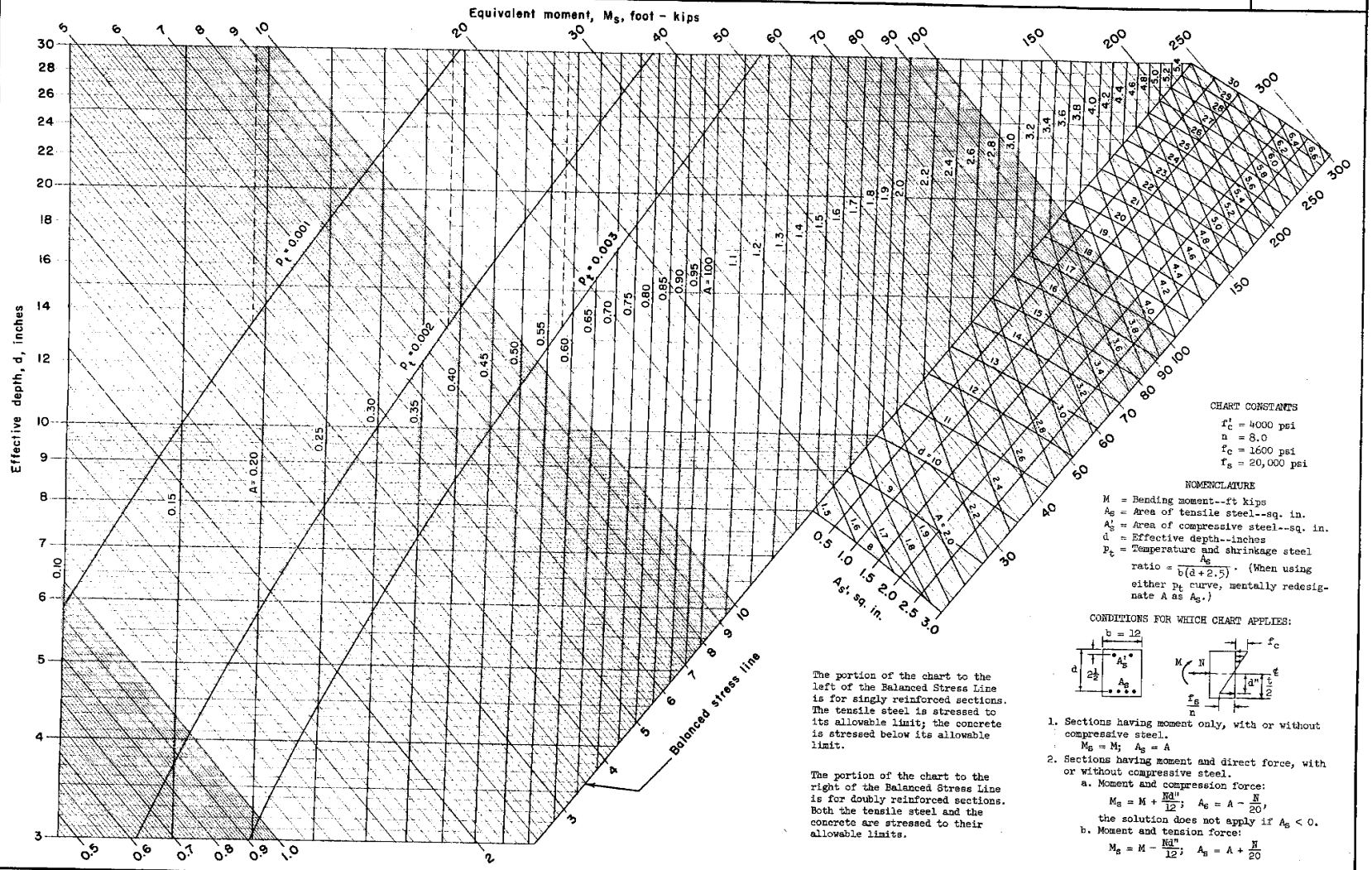
REFERENCE
ACI Building Code (ACI 318-63)

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES-163
SHEET 3 OF 3
DATE 7-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN BENDING ONLY OR BENDING COMBINED WITH DIRECT FORCE. FOR 12 INCH WIDTH

f'_c = 4000 psi



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The portion of the chart to the right of the Balanced Stress Line is for doubly reinforced sections. Both the tensile steel and the concrete are stressed to their allowable limits.

CHART CONSTANTS

f'_c = 4000 psi
n = 8.0
f_c = 1600 psi
f_s = 20,000 psi

NOMENCLATURE

M = Bending moment--ft kips
A_s = Area of tensile steel--sq. in.
A'_s = Area of compressive steel--sq. in.
d = Effective depth--inches
p_t = Temperature and shrinkage steel ratio = $\frac{A_s}{b(d+2.5)}$. (When using either p_t curve, mentally redesignate A as A_s.)

CONDITIONS FOR WHICH CHART APPLIES:

1. Sections having moment only, with or without compressive steel.
   $M_s = M$; $A_s = A$
2. Sections having moment and direct force, with or without compressive steel.
   a. Moment and compression force:
   $M_s = M + \frac{Nd''}{12}$; $A_s = A - \frac{N}{20}$,
   the solution does not apply if $A_s < 0$.
   b. Moment and tension force:
   $M_s = M - \frac{Nd''}{12}$; $A_s = A + \frac{N}{20}$

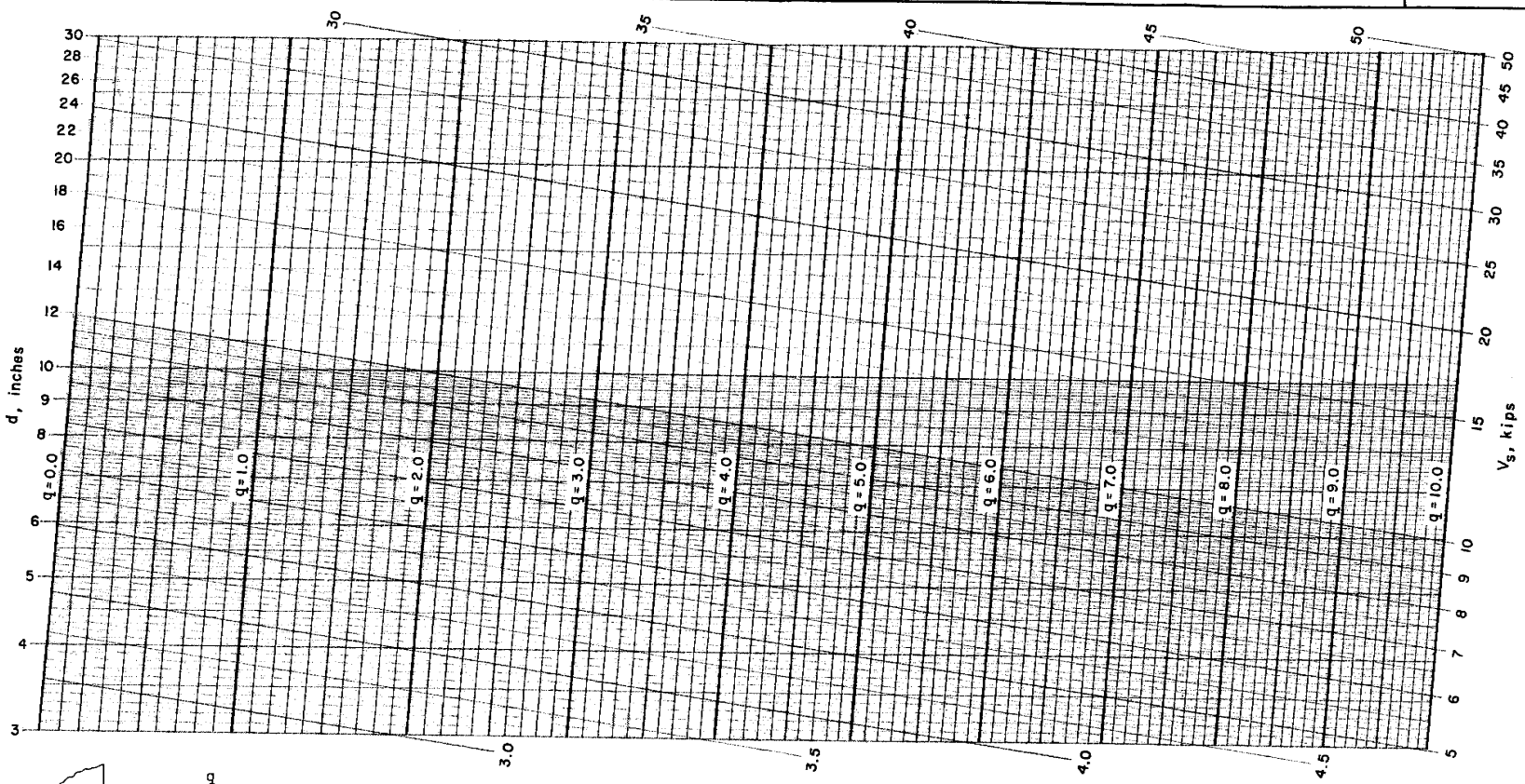
REFERENCE

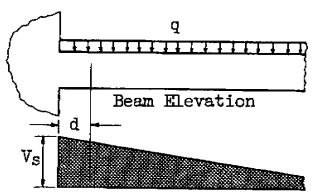
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO. ES- 164
SHEET 1 OF 3
DATE 4-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
ALLOWABLE TOTAL SHEAR AT FACE OF SUPPORT OF UNIFORMLY LOADED
BEAM WITHOUT WEB REINFORCEMENT. FOR 12 INCH WIDTH.

$f'_c = 4000$ psi





$$V_s = \frac{12 v_c d}{1000}\left(1 + \frac{1000q}{144 v_c}\right)$$

V_s = allowable total shear at face of support, kips
v_c = allowable shear stress at distance equal to [d] from face of support = $1.1\sqrt{f'_c}$, psi
d = effective depth, inches
q = unit load, kips per lineal foot

Solution is valid if, within the interval [d]:
(1) there is no concentrated load, and
(2) the unit load is constant.

REFERENCE
ACI Building Code (ACI 318-63)

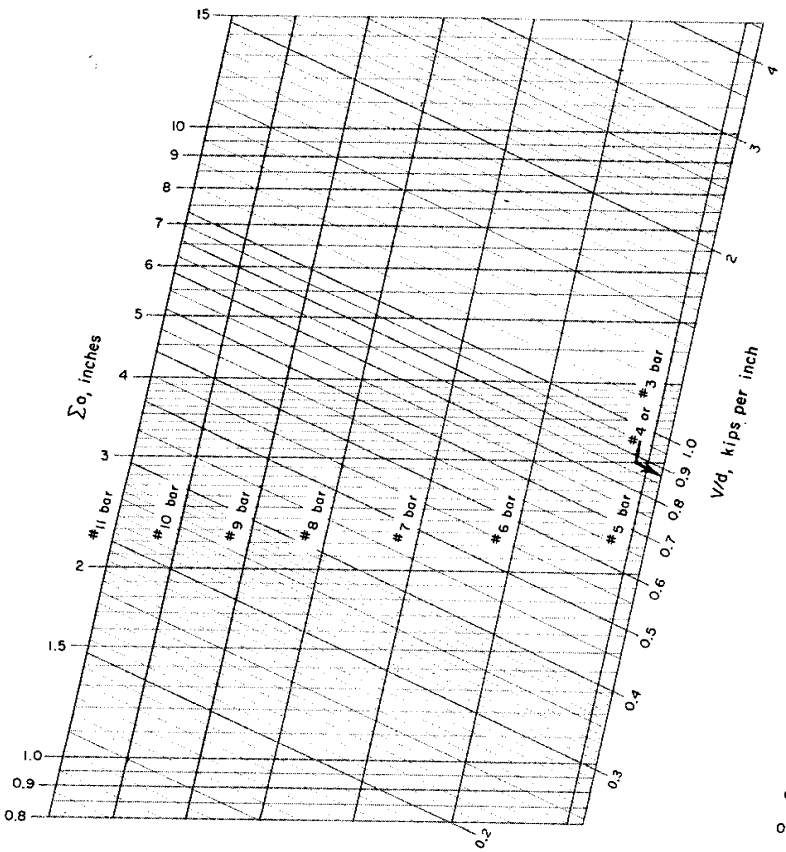
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 164
SHEET 2 OF 3
DATE 7-64

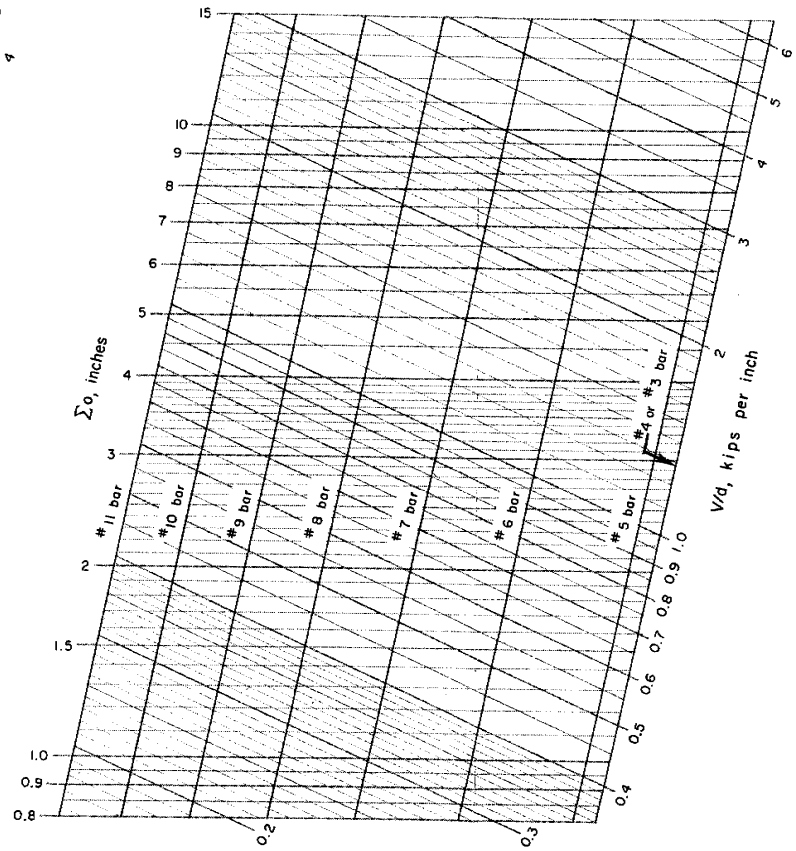
STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
PERIMETER OF TENSILE REINFORCEMENT REQUIRED FOR FLEXURAL BOND.
FOR 12 INCH WIDTH

$f'_c = 4000$ psi

Tension Top Bars



Tension Bars Other Than Top Bars



$$\Sigma o = \frac{8000}{7u}(V/d)$$

Σo = required perimeter of tensile reinforcement at the section, inches
V = total shear at the section, kips
d = effective depth, inches

u = allowable bond stress, psi
For tension top bars $u = 3.4\sqrt{f'_c}/D \leq 350$
For tension bars other than top bars $u = 4.8\sqrt{f'_c}/D \leq 500$

REFERENCE

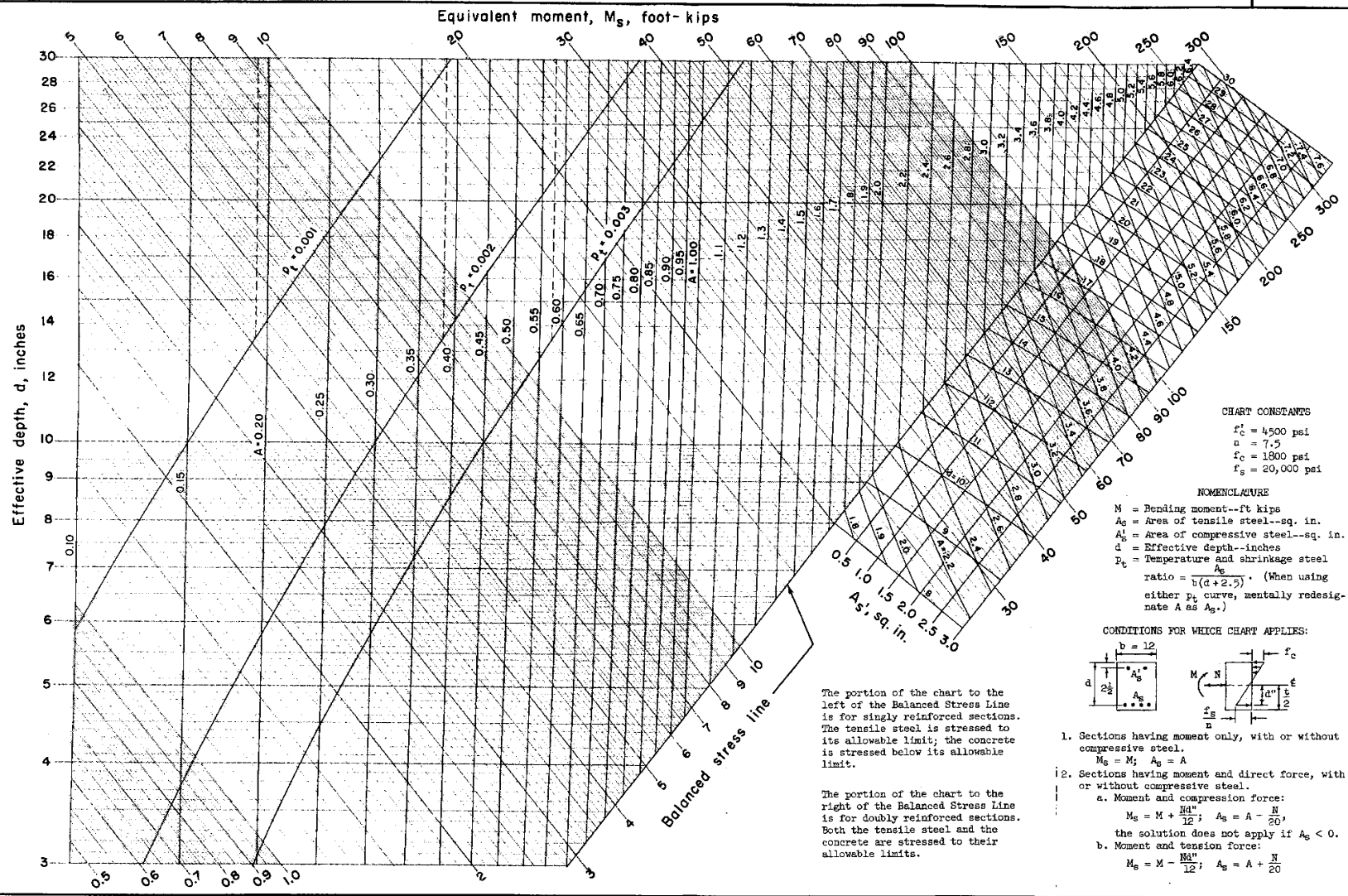
ACI Building Code (ACI 318-63)

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES-164
SHEET 3 OF 3
DATE 7-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN BENDING ONLY OR BENDING COMBINED WITH DIRECT FORCE. FOR 12 INCH WIDTH

f'_c = 4500 psi



The portion of the chart to the left of the Balanced Stress Line is for singly reinforced sections. The tensile steel is stressed to its allowable limit; the concrete is stressed below its allowable limit.

The portion of the chart to the right of the Balanced Stress Line is for doubly reinforced sections. Both the tensile steel and the concrete are stressed to their allowable limits.

CHART CONSTANTS

f'_c = 4500 psi
n = 7.5
f_c = 1800 psi
f_s = 20,000 psi

NOMENCLATURE

M = Bending moment--ft kips
A_s = Area of tensile steel--sq. in.
A'_s = Area of compressive steel--sq. in.
d = Effective depth--inches
p_t = Temperature and shrinkage steel ratio = $\frac{A_s}{b(d+2.5)}$. (When using either p_t curve, mentally redesignate A as A_s.)

CONDITIONS FOR WHICH CHART APPLIES:

1. Sections having moment only, with or without compressive steel.
   $M_s = M$; $A_s = A$
2. Sections having moment and direct force, with or without compressive steel.
   a. Moment and compression force:
   $M_s = M + \frac{Nd''}{12}$; $A_s = A - \frac{N}{20}$,
   the solution does not apply if $A_s < 0$.
   b. Moment and tension force:
   $M_s = M - \frac{Nd''}{12}$; $A_s = A + \frac{N}{20}$

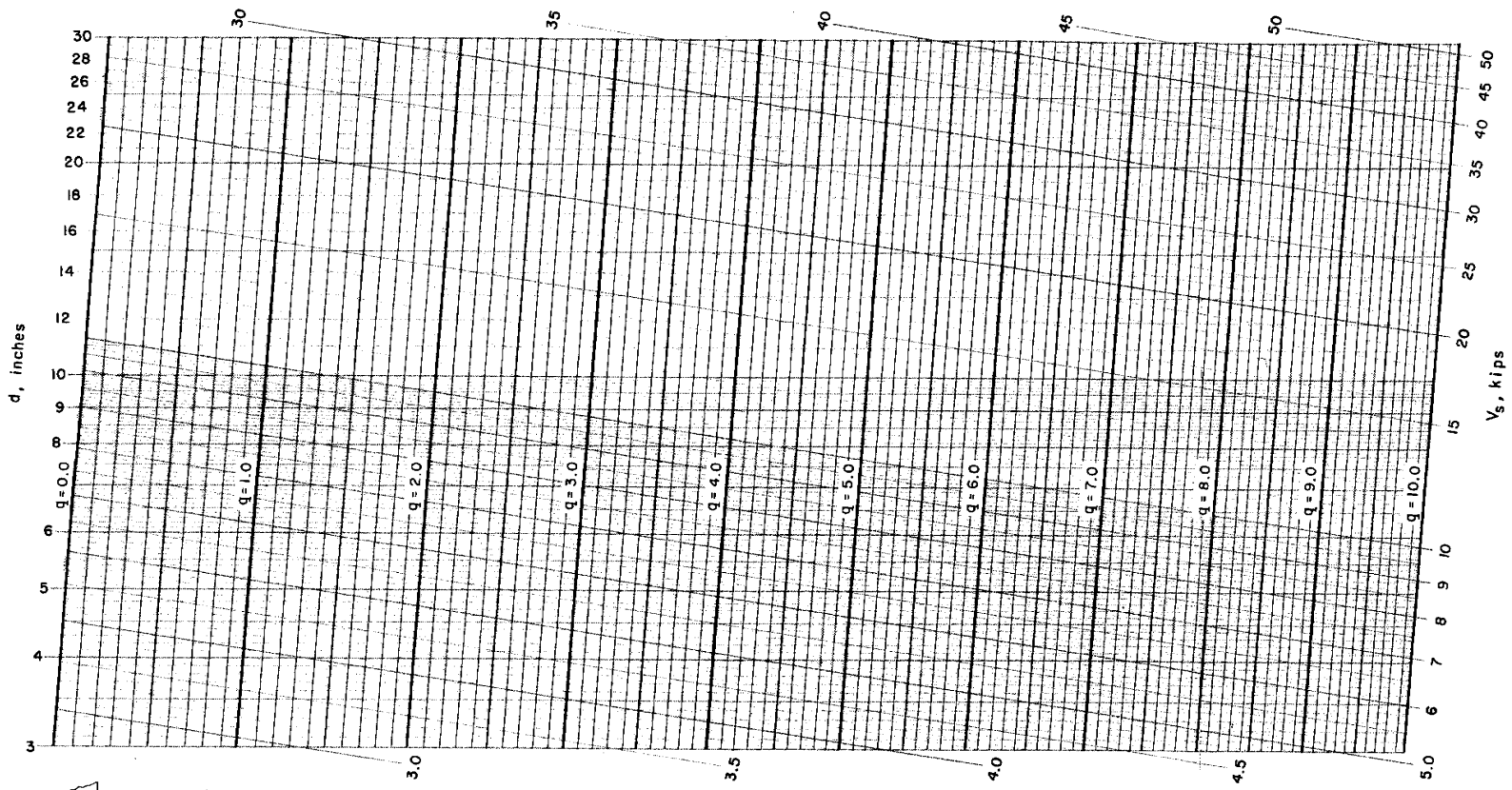
REFERENCE

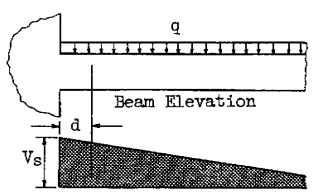
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION · DESIGN SECTION

STANDARD DWG. NO. ES- 165
SHEET 1 OF 3
DATE 4-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
ALLOWABLE TOTAL SHEAR AT FACE OF SUPPORT OF UNIFORMLY LOADED
BEAM WITHOUT WEB REINFORCEMENT. FOR 12 INCH WIDTH.

$f'_c = 4500$ psi





Allowable Shear Diagram

$$V_s = \frac{12 v_c d}{1000}\left(1 + \frac{1000q}{144 v_c}\right)$$

V_s = allowable total shear at face of support, kips
v_c = allowable shear stress at distance equal to [d] from face of support = $1.1\sqrt{f'_c}$. psi
d = effective depth, inches
q = unit load, kips per lineal foot

Solution is valid if, within the interval [d]:
(1) there is no concentrated load, and
(2) the unit load is constant.

REFERENCE
ACI Building Code (ACI 318-63)

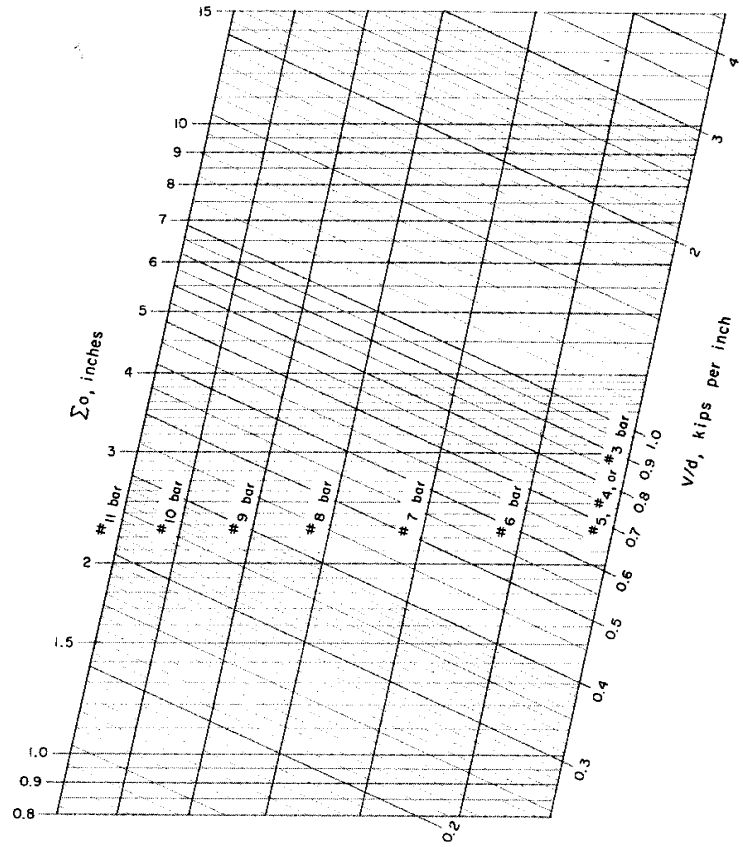
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 165
SHEET 2 OF 3
DATE 7-64

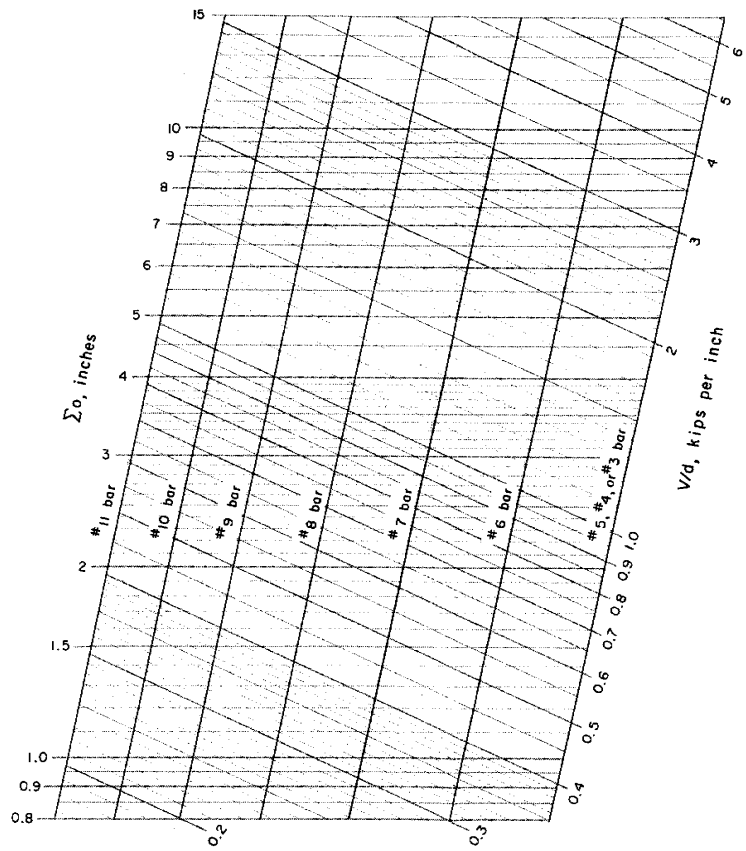
STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
PERIMETER OF TENSILE REINFORCEMENT REQUIRED FOR FLEXURAL BOND.
FOR 12 INCH WIDTH

$f'_c = 4500$ psi

Tension Top Bars



Tension Bars Other Than Top Bars



$$\Sigma o = \frac{8000}{7u}(V/d)$$

Σo = required perimeter of tensile reinforcement at the section, inches
V = total shear at the section, kips
d = effective depth, inches

u = allowable bond stress, psi

For tension top bars $u = 3.4\sqrt{f'_c}/D \leq 350$

For tension bars other than top bars $u = 4.8\sqrt{f'_c}/D \leq 500$

REFERENCE

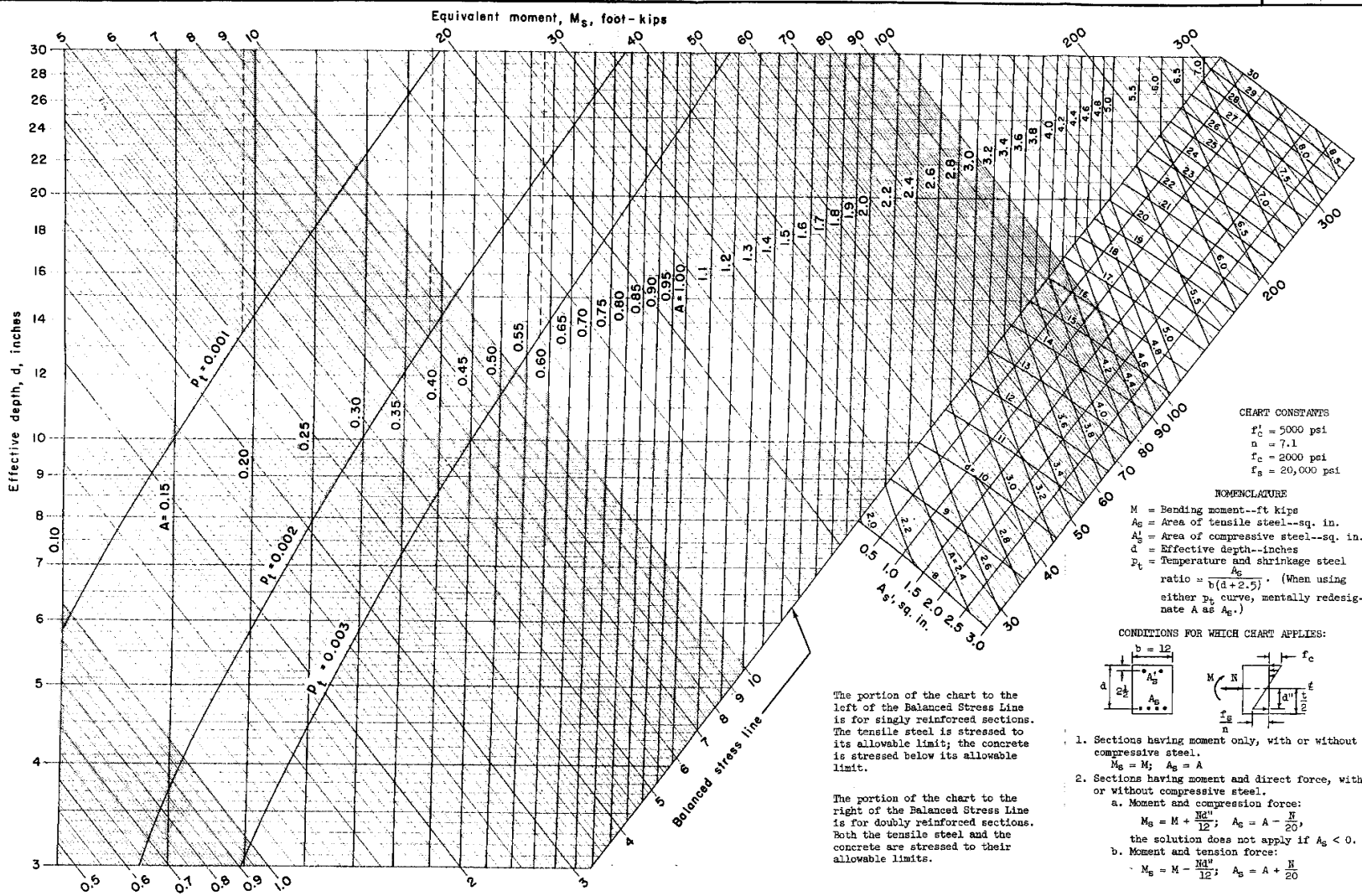
ACI Building Code (ACI 318 - 63)

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES 165
SHEET 3 OF 3
DATE 7-61

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN
BENDING ONLY OR BENDING COMBINED WITH DIRECT FORCE.
FOR 12 INCH WIDTH

f'_c = 5000 psi



The portion of the chart to the left of the Balanced Stress Line is for singly reinforced sections. The tensile steel is stressed to its allowable limit; the concrete is stressed below its allowable limit.

The portion of the chart to the right of the Balanced Stress Line is for doubly reinforced sections. Both the tensile steel and the concrete are stressed to their allowable limits.

CHART CONSTANTS

f'_c = 5000 psi
n = 7.1
f_c = 2000 psi
f_s = 20,000 psi

NOMENCLATURE

M = Bending moment--ft kips
A_s = Area of tensile steel--sq. in.
A'_s = Area of compressive steel--sq. in.
d = Effective depth--inches
p_t = Temperature and shrinkage steel ratio = $\frac{A_s}{b(d+2.5)}$. (When using either p_t curve, mentally redesignate A as A_s.)

CONDITIONS FOR WHICH CHART APPLIES:

1. Sections having moment only, with or without compressive steel.
   $M_s = M$; $A_s = A$
2. Sections having moment and direct force, with or without compressive steel.
   a. Moment and compression force:
   $M_s = M + \frac{Nd''}{12}$; $A_s = A - \frac{N}{20}$,
   the solution does not apply if $A_s < 0$.
   b. Moment and tension force:
   $M_s = M - \frac{Nd''}{12}$; $A_s = A + \frac{N}{20}$

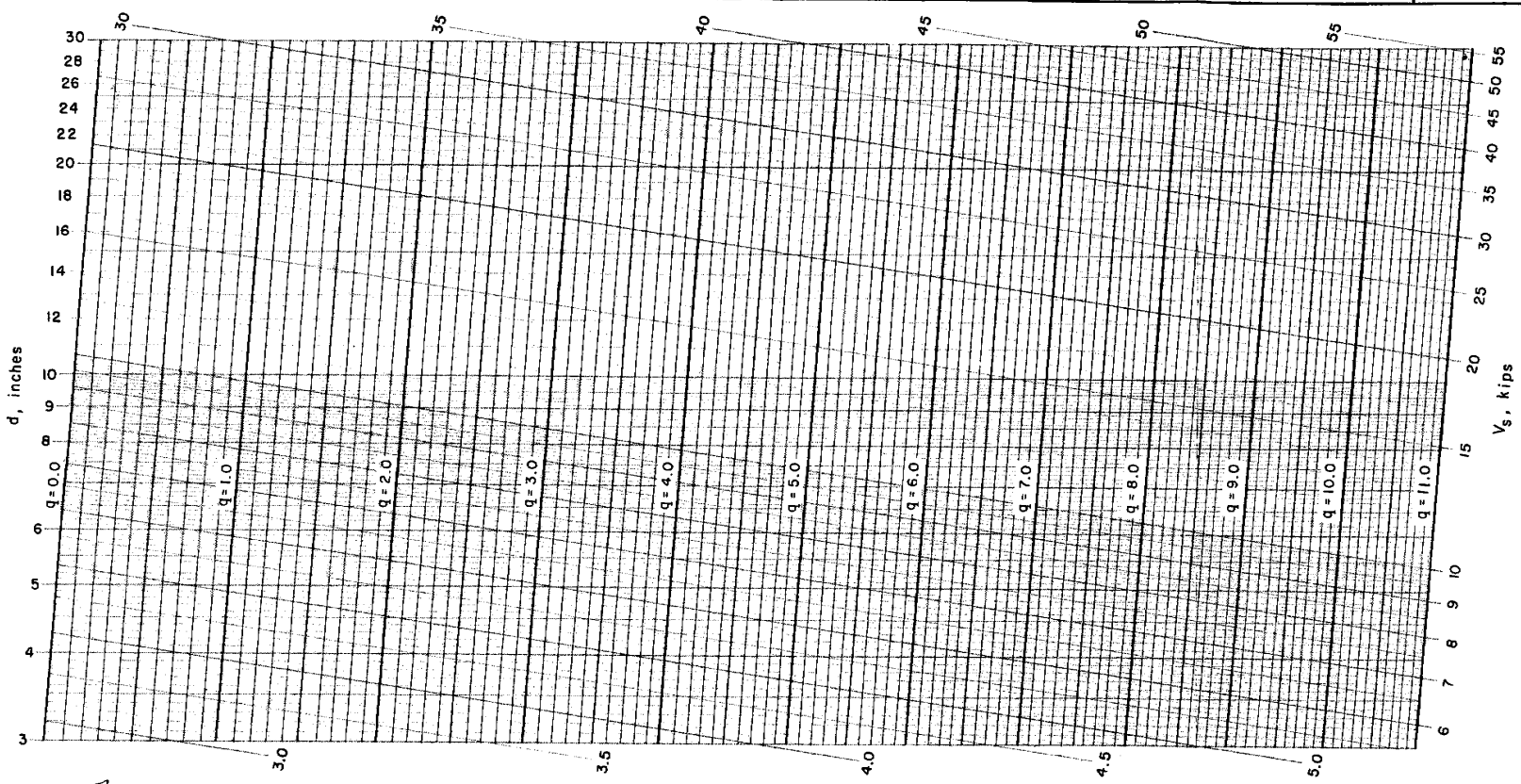
REFERENCE

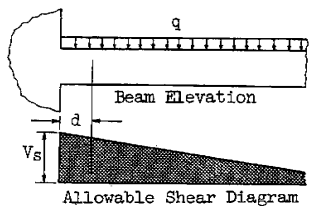
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 166
SHEET 1 OF 3
DATE 4-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
ALLOWABLE TOTAL SHEAR AT FACE OF SUPPORT OF UNIFORMLY LOADED
BEAM WITHOUT WEB REINFORCEMENT. FOR 12 INCH WIDTH.

$f'_c = 5000$ psi





$$V_s = \frac{12 v_c d}{1000}\left(1 + \frac{1000q}{144 v_c}\right)$$

V_s = allowable total shear at face of support, kips
v_c = allowable shear stress at distance equal to [d] from face of support = $1.1\sqrt{f'_c}$, psi
d = effective depth, inches
q = unit load, kips per lineal foot

Solution is valid if, within the interval [d]:
(1) there is no concentrated load, and
(2) the unit load is constant.

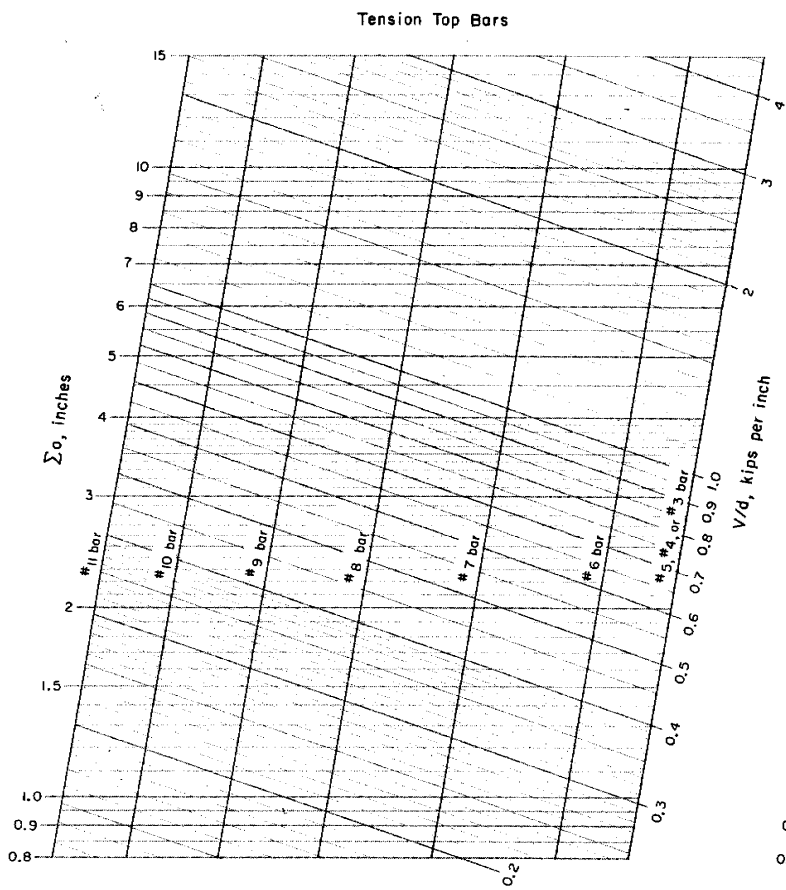
REFERENCE
ACI Building Code [ACI 318-63]

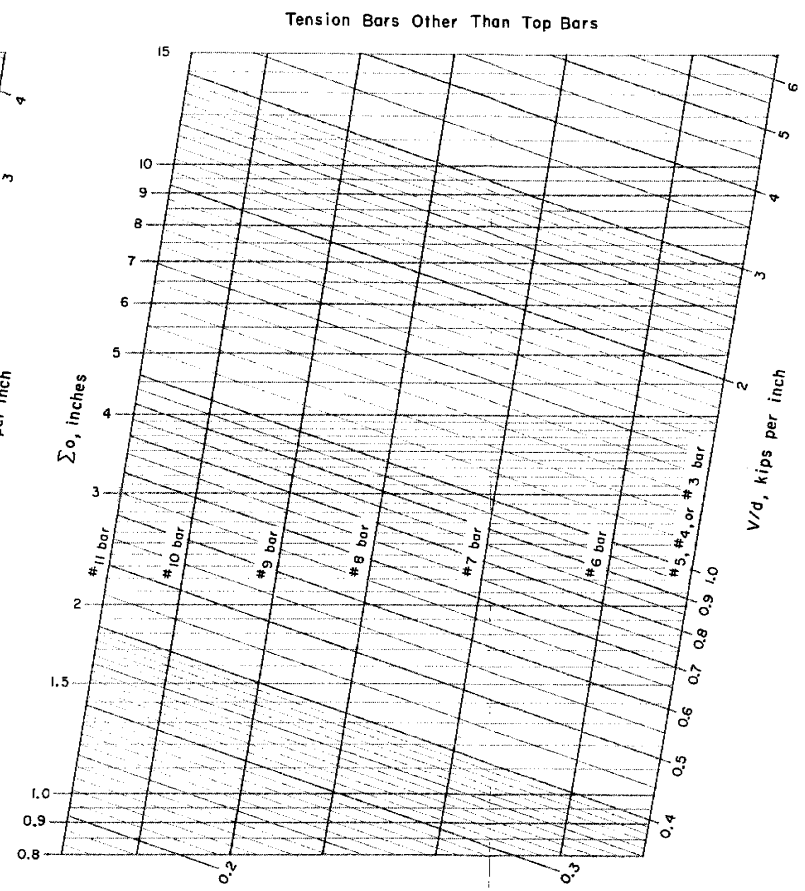
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 166
SHEET 2 OF 3
DATE 7-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
PERIMETER OF TENSILE REINFORCEMENT REQUIRED FOR FLEXURAL BOND.
FOR 12 INCH WIDTH

$f'_c = 5000$ psi





$$\Sigma o = \frac{8000}{7u}(V/d)$$

Σo = required perimeter of tensile reinforcement at the section, inches
V = total shear at the section, kips
d = effective depth, inches

u = allowable bond stress, psi
For tension top bars $u = 3.4\sqrt{f'_c}/D \leq 350$
For tension bars other than top bars $u = 4.8\sqrt{f'_c}/D \leq 500$

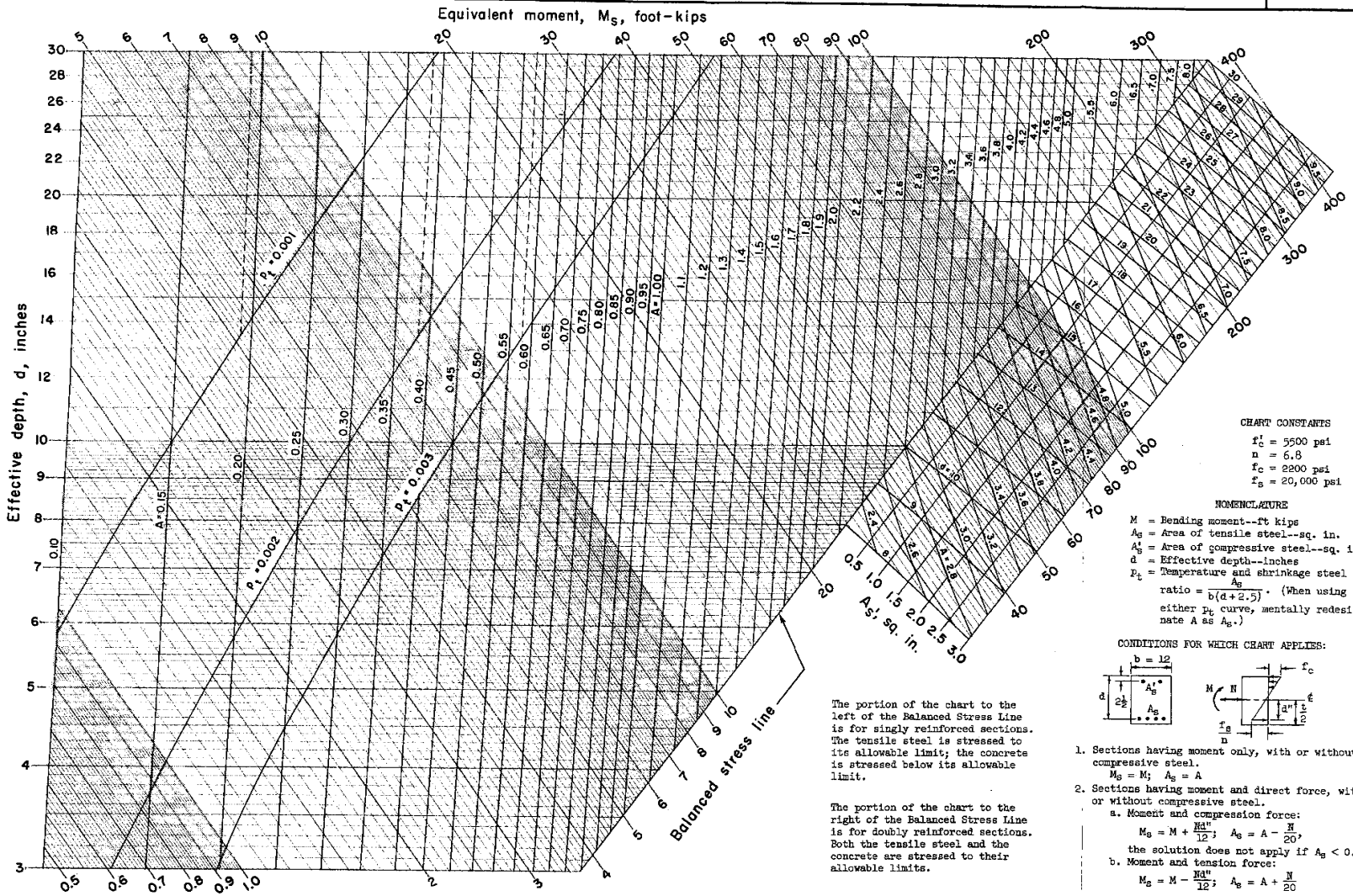
REFERENCE
ACI Building Code (ACI 318 - 63)

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 166
SHEET 3 OF 3
DATE 7-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN
BENDING ONLY OR BENDING COMBINED WITH DIRECT FORCE.
FOR 12 INCH WIDTH

$f'_c = 5500$ psi



The portion of the chart to the left of the Balanced Stress Line is for singly reinforced sections. The tensile steel is stressed to its allowable limit; the concrete is stressed below its allowable limit.

The portion of the chart to the right of the Balanced Stress Line is for doubly reinforced sections. Both the tensile steel and the concrete are stressed to their allowable limits.

CHART CONSTANTS

f'_c = 5500 psi
n = 6.8
f_c = 2200 psi
f_s = 20,000 psi

NOMENCLATURE

M = Bending moment--ft kips
A_s = Area of tensile steel--sq. in.
A'_s = Area of compressive steel--sq. in.
d = Effective depth--inches
P_t = Temperature and shrinkage steel ratio = $\frac{A_s}{b(d+2.5)}$. (When using either P_t curve, mentally redesignate A as A_s.)

CONDITIONS FOR WHICH CHART APPLIES:

1. Sections having moment only, with or without compressive steel.
   $M_s = M$; $A_s = A$
2. Sections having moment and direct force, with or without compressive steel.
   a. Moment and compression force:
   $M_s = M + \frac{Nd''}{12}$; $A_s = A - \frac{N}{20}$,
   the solution does not apply if $A_s < 0$.
   b. Moment and tension force:
   $M_s = M - \frac{Nd''}{12}$; $A_s = A + \frac{N}{20}$

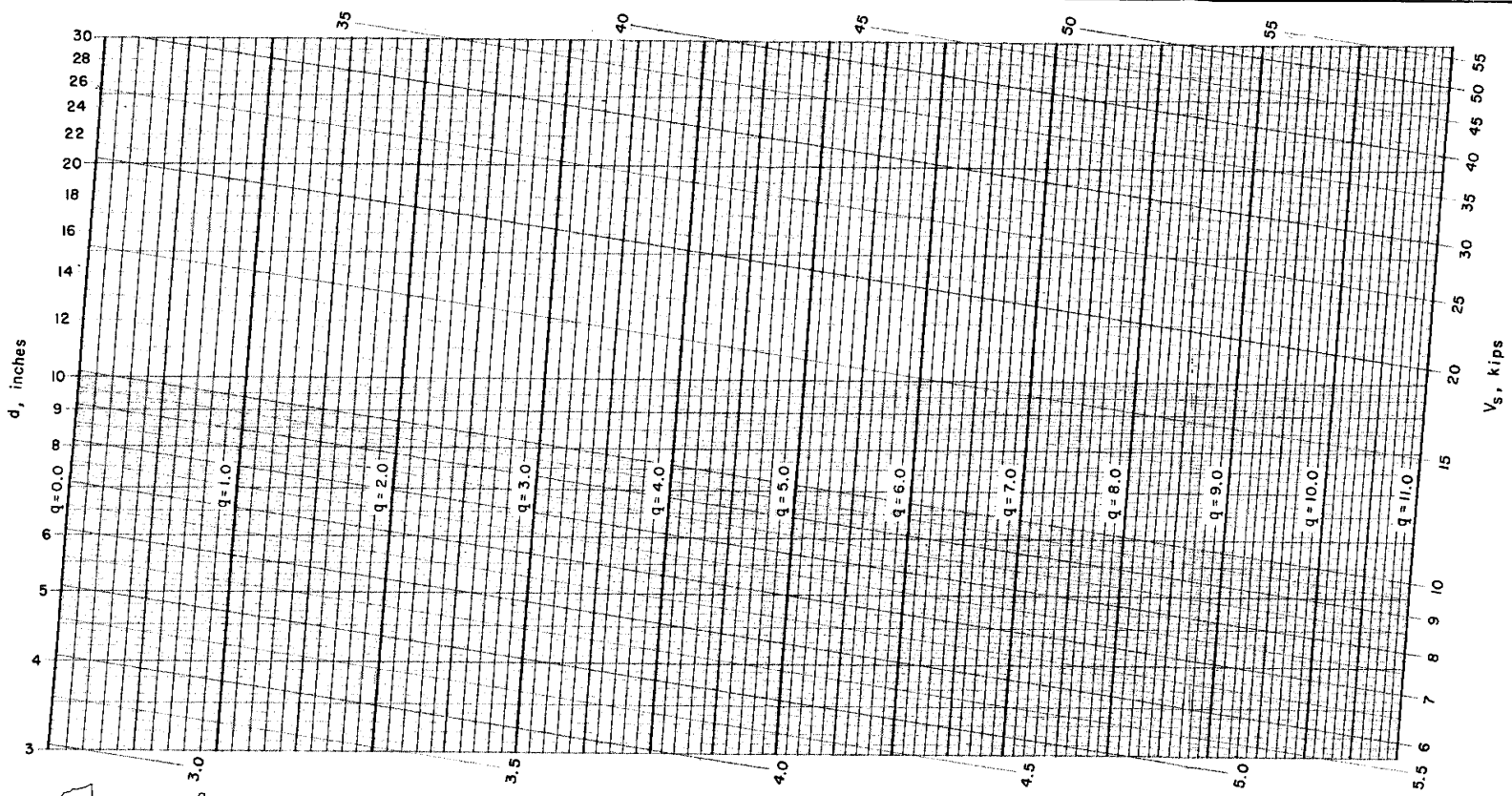
REFERENCE

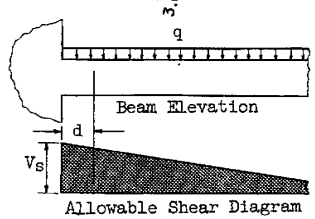
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 167
SHEET 1 OF 3
DATE 4-54

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
ALLOWABLE TOTAL SHEAR AT FACE OF SUPPORT OF UNIFORMLY LOADED BEAM WITHOUT WEB REINFORCEMENT. FOR 12 INCH WIDTH.

$f'_c = 5500$ psi





$$V_s = \frac{12 v_c d}{1000}\left(1 + \frac{1000q}{144 v_c}\right)$$

V_s = allowable total shear at face of support, kips
v_c = allowable shear stress at distance equal to [d] from face of support = $1.1\sqrt{f'_c}$, psi
d = effective depth, inches
q = unit load, kips per lineal foot

Solution is valid if, within the interval [d]:
(1) there is no concentrated load, and
(2) the unit load is constant.

REFERENCE
ACI Building Code (ACI 318-63)

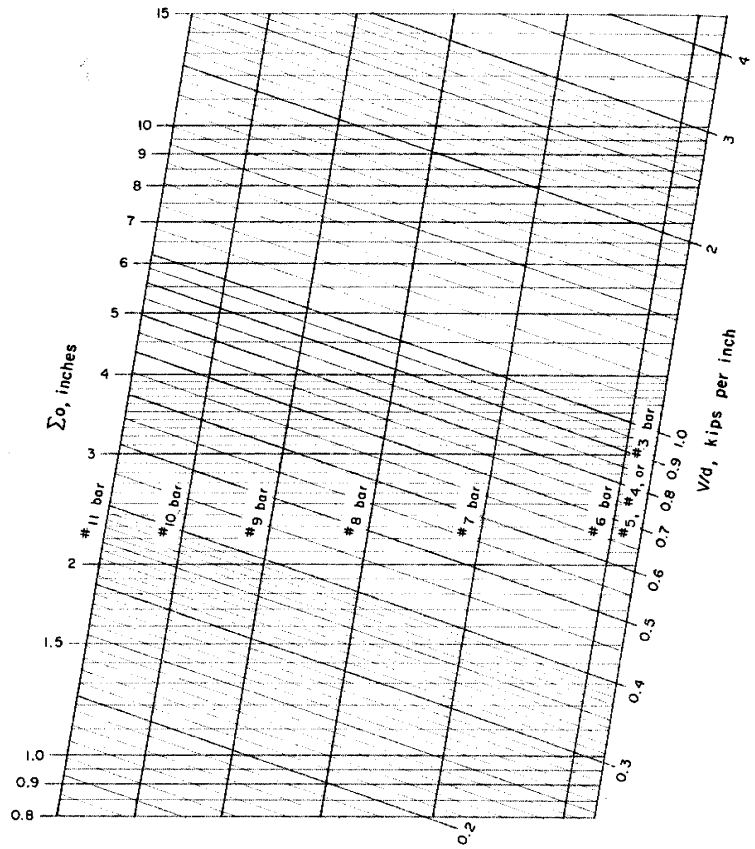
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 167
SHEET 2 OF 3
DATE 7-64

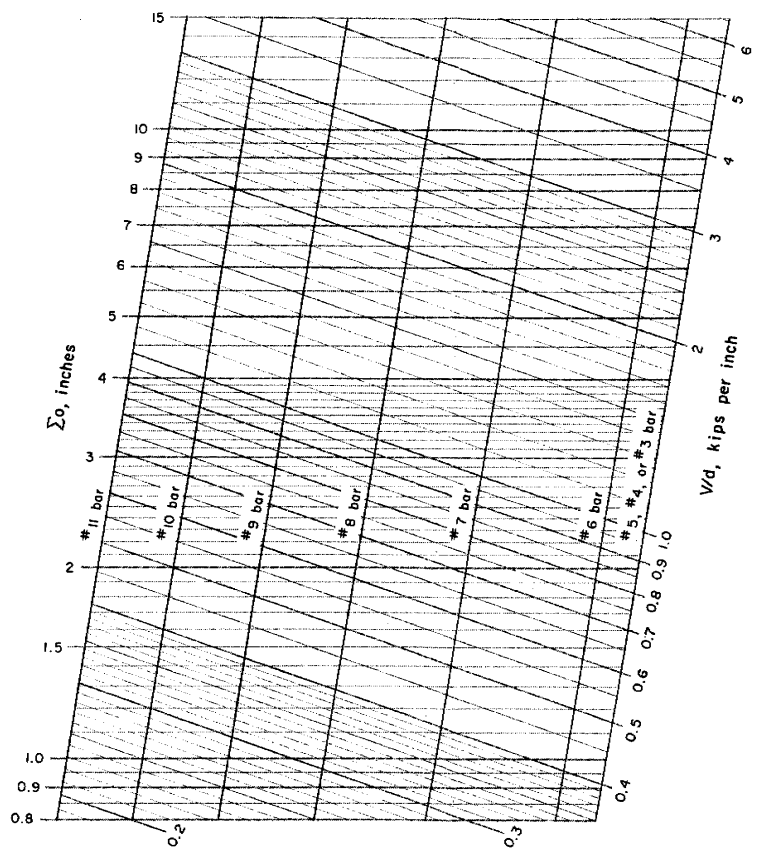
STRUCTURAL DESIGN : REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
PERIMETER OF TENSILE REINFORCEMENT REQUIRED FOR FLEXURAL BOND.
FOR 12 INCH WIDTH

$f'_c = 5500$ psi

Tension Top Bars



Tension Bars Other Than Top Bars



$$\Sigma o = \frac{8000}{7u}(V/d)$$

Σo = required perimeter of tensile reinforcement at the section, inches
V = total shear at the section, kips
d = effective depth, inches

u = allowable bond stress, psi
For tension top bars $u = 3.4\sqrt{f'_c}/D \leq 350$
For tension bars other than top bars $u = 4.8\sqrt{f'_c}/D \leq 500$

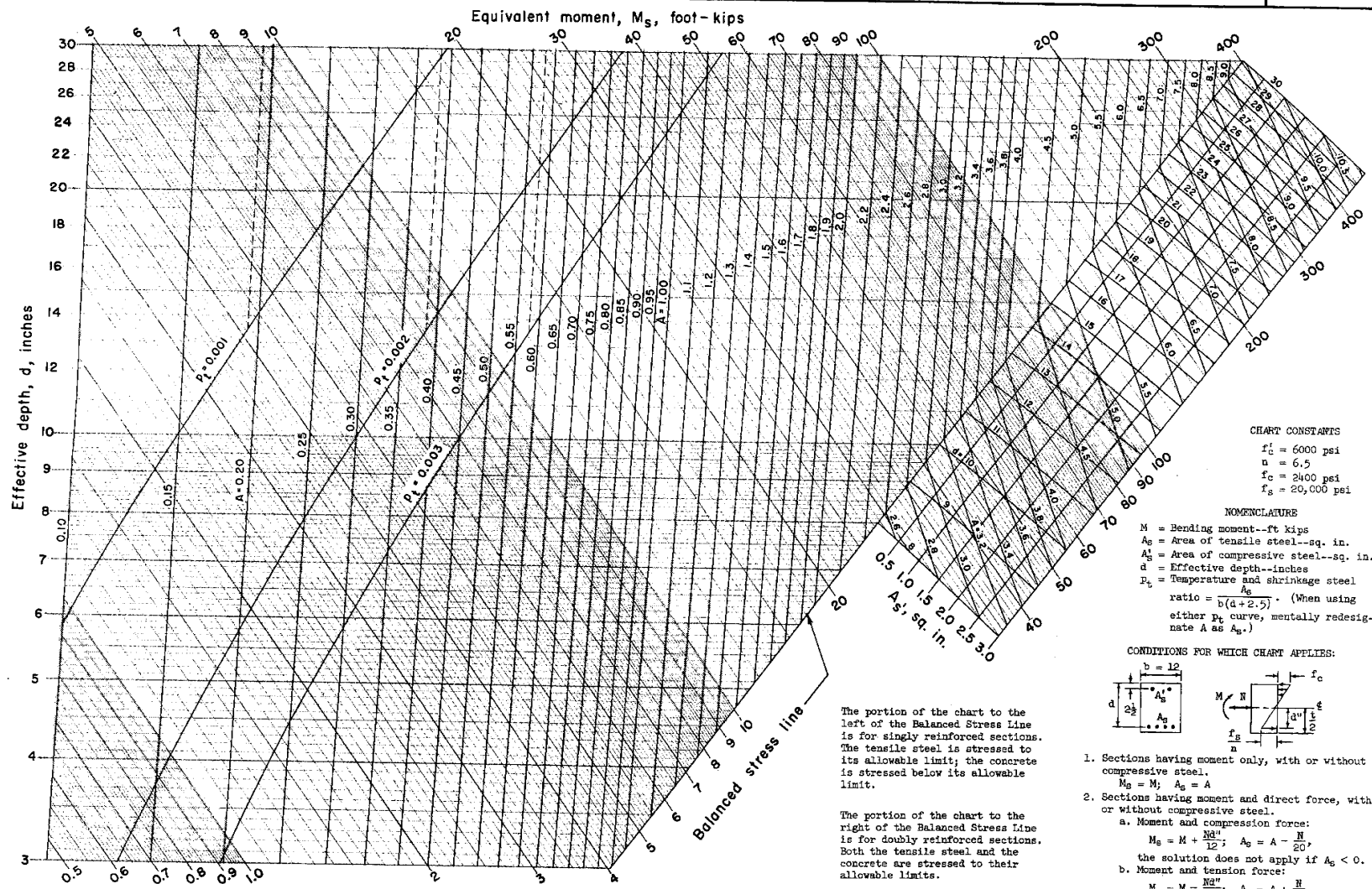
REFERENCE
ACI Building Code (ACI 318-63)

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES-167
SHEET 3 OF 3
DATE 7-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN BENDING ONLY OR BENDING COMBINED WITH DIRECT FORCE. FOR 12 INCH WIDTH

$f'_c = 6000$ psi



The portion of the chart to the left of the Balanced Stress Line is for singly reinforced sections. The tensile steel is stressed to its allowable limit; the concrete is stressed below its allowable limit.

The portion of the chart to the right of the Balanced Stress Line is for doubly reinforced sections. Both the tensile steel and the concrete are stressed to their allowable limits.

CHART CONSTANTS

$f'_c = 6000$ psi
$n = 6.5$
$f_c = 2400$ psi
$f_s = 20{,}000$ psi

NOMENCLATURE

M = Bending moment--ft kips
A_s = Area of tensile steel--sq. in.
A'_s = Area of compressive steel--sq. in.
d = Effective depth--inches
P_t = Temperature and shrinkage steel ratio $= \frac{A_s}{b(d+2.5)}$. (When using either P_t curve, mentally redesignate A as A_s.)

CONDITIONS FOR WHICH CHART APPLIES:

1. Sections having moment only, with or without compressive steel.
   $M_s = M$; $A_s = A$
2. Sections having moment and direct force, with or without compressive steel.
   a. Moment and compression force:
   $M_s = M + \frac{Nd''}{12}$; $A_s = A - \frac{N}{20}$,
   the solution does not apply if $A_s < 0$.
   b. Moment and tension force:
   $M_s = M - \frac{Nd''}{12}$; $A_s = A + \frac{N}{20}$

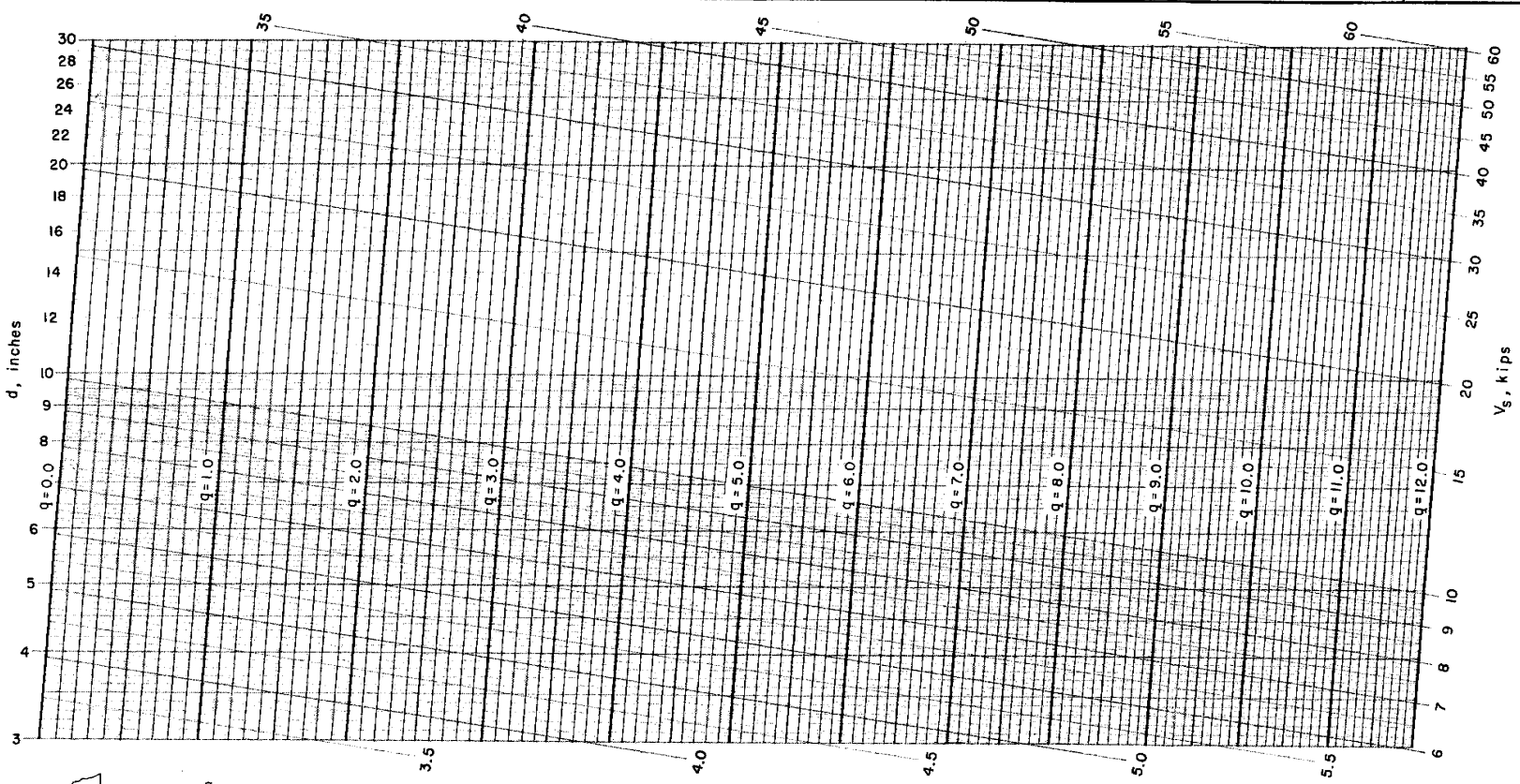
REFERENCE

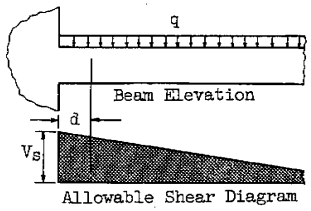
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO. ES- 168
SHEET 1 OF 3
DATE 4-54

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
ALLOWABLE TOTAL SHEAR AT FACE OF SUPPORT OF UNIFORMLY LOADED BEAM WITHOUT WEB REINFORCEMENT. FOR 12 INCH WIDTH.

f'_c=6000 psi





$$V_s = \frac{12 v_c d}{1000}\left(1 + \frac{1000q}{144 v_c}\right)$$

V_s = allowable total shear at face of support, kips
v_c = allowable shear stress at distance equal to [d] from face of support = $1.1\sqrt{f'_c}$, psi
d = effective depth, inches
q = unit load, kips per lineal foot

Solution is valid if, within the interval [d]:
(1) there is no concentrated load, and
(2) the unit load is constant.

REFERENCE
ACI Building Code (ACI 318-63)

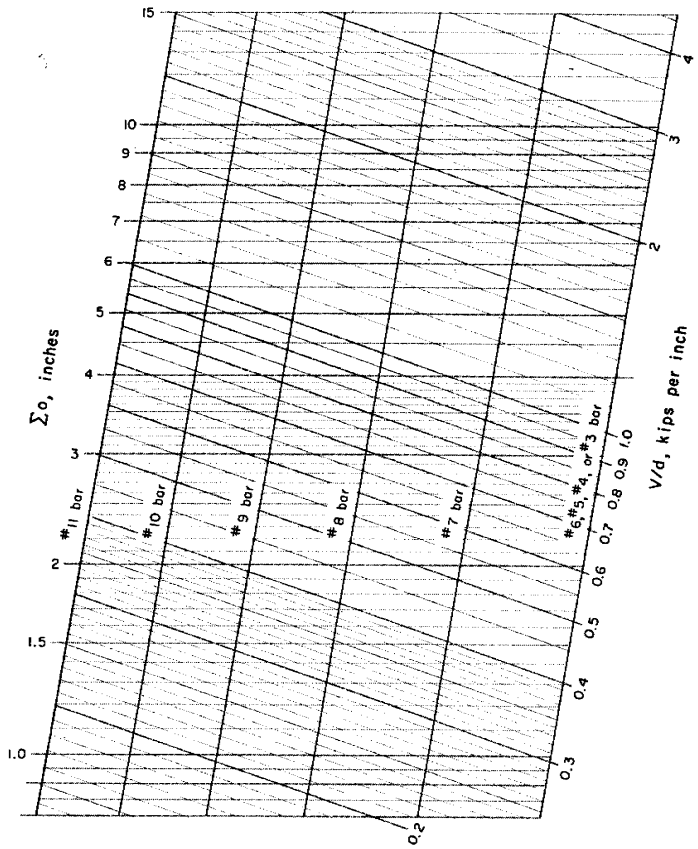
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 168
SHEET 2 OF 3
DATE 7-64

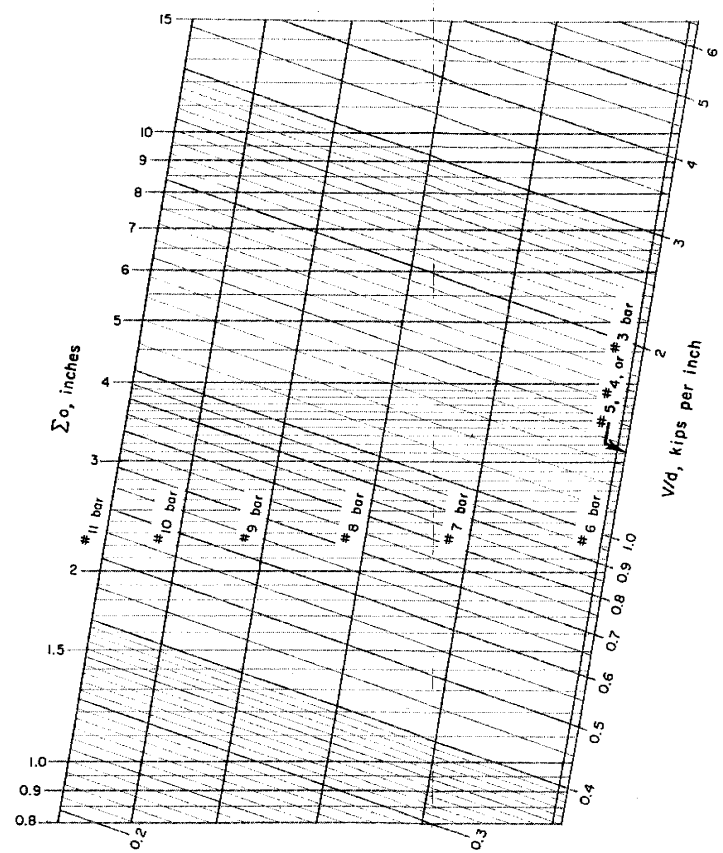
STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN, WORKING STRESS DESIGN.
PERIMETER OF TENSILE REINFORCEMENT REQUIRED FOR FLEXURAL BOND.
FOR 12 INCH WIDTH

$f'_c = 6000$ psi

Tension Top Bars



Tension Bars Other Than Top Bars



$$\Sigma o = \frac{8000}{7u}(V/d)$$

Σo = required perimeter of tensile reinforcement at the section, inches
V = total shear at the section, kips
d = effective depth, inches

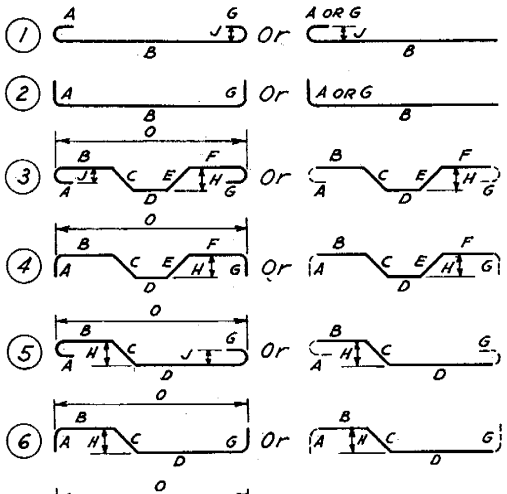
u = allowable bond stress, psi
For tension top bars $u = 3.4\sqrt{f'_c}/D \leq 350$
For tension bars other than top bars $u = 4.8\sqrt{f'_c}/D \leq 500$

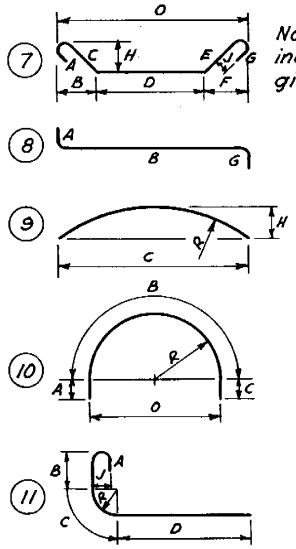
REFERENCE
ACI Building Code (ACI 318-63)

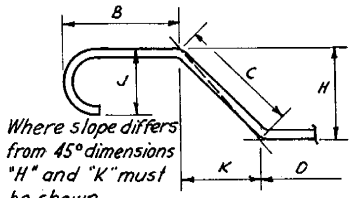
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES- 168
SHEET 3 OF 3
DATE 7-64

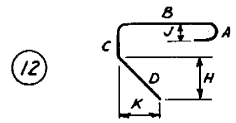
STRUCTURAL DESIGN: REINFORCED CONCRETE
TYPICAL BAR TYPES

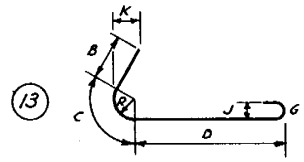


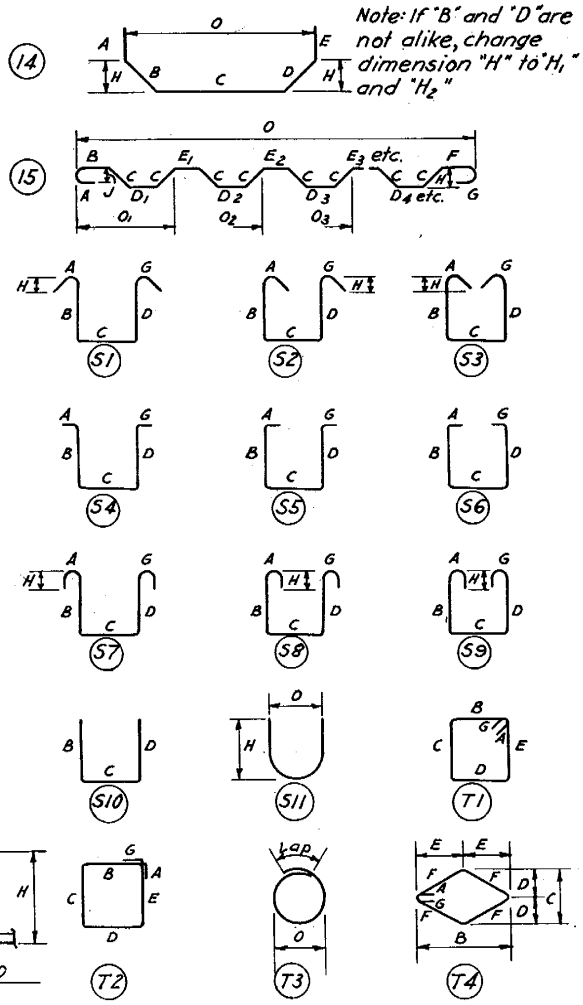




ENLARGED VIEW SHOWING BAR BENDING DETAILS







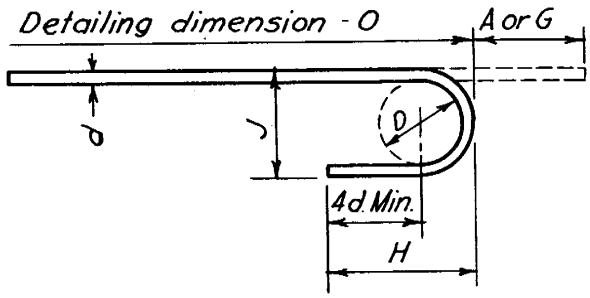
NOTES

1. All dimensions are out to out of bar.
2. "J" dimension on 180° hooks to be shown only where necessary to restrict hook size. Otherwise, standard hooks are to be used.
3. Where "J" is not shown, "J" will be kept equal to or less than "H." Where "J" can exceed "H", it should be shown.
4. "H" dimension on stirrups to be shown where necessary to restrict hooks.
5. Where bars are to be bent more accurately than standard bending tolerances, bending dimensions which require closer working should have limits indicated.
6. Figures in circles show types.

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING STANDARDS UNIT	STANDARD DWG. NO. ES-18 SHEET 1 OF 1 DATE 4-12-50
ACI Standard 315-48		

STRUCTURAL DESIGN: REINFORCED CONCRETE
STANDARD HOOK DETAILS, ACI CODE

RECOMMENDED SIZES OF 180° HOOK



D = 6 d for #3 thru #7
D = 8 d for #8 thru #11

Bar Size	Hook A or G	J	Approx. H
#3	4	3	3
#4	6	4	4 1/2
#5	7	5	5
#6	8	6	5 3/4
#7	10	7	7 1/2
#8	1-1	10	9
#9	1-3	11 1/4	10 1/4
#10	1-5	1-0 1/2	11 3/4
#11	1-8	1-2 1/4	1-2 1/4

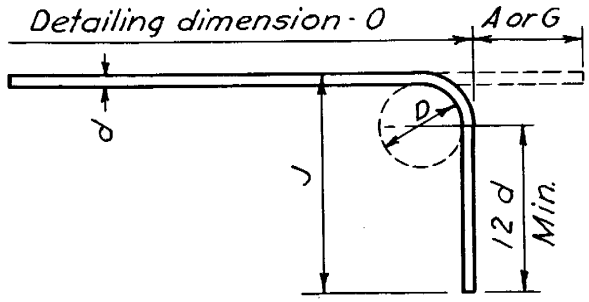
MINIMUM SIZES OF 180° HOOK

D = 5 d Min.
D = 11 d Max.

NOTE: This table to be used only for special conditions where hook smaller than recommended sizes is necessary. Not appropriate for hard grades of steel.

Bar Size	Hook A or G	J	Approx. H
#3	4	2 3/4	3
#4	5	3 1/2	3 3/4
#5	6	4 1/2	4 1/2
#6	8	5 1/4	6 1/4
#7	9	6 1/4	6 3/4
#8	10	7	7 1/2
#9	12	8	9 1/4
#10	1-1	8 3/4	10
#11	1-2	10	10 3/4

RECOMMENDED SIZES & MINIMUM SIZES OF 90° HOOK



D = 7 d

Bar Size	Hook A or G	Approx. J
#3	5	6
#4	7	8 1/4
#5	9	10 3/4
#6	10	1-0
#7	1-0	1-2 1/4
#8	1-2	1-4 3/4
#9	1-4	1-7
#10	1-6	1-9 1/2
#11	1-8	2-0

REFERENCE
ACI Standard 315-48

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING STANDARDS UNIT

STANDARD DWG. NO. ES - 20
SHEET 1 OF 1
DATE 4-18-50

Revised 12-22-50

"(d) Do's and Don'ts. Experienced designers and detailers will develop a list of "do's and don'ts" such as the following:

1. Give complete and accurate dimensions on engineering drawings.

2. Show details at corners and intersections of walls and at window and door openings.

3. Use notations which cannot be misinterpreted, such as "each way" rather than "both ways";"No. 5 at 12, each face staggered" rather than "No. 5 at 12 staggered."

4. Congestion of steel should be avoided at points where members intersect. Make certain that all reinforcement shown can be properly placed. For example, at the intersection of a beam and girder, the beam bars should be bent at a different elevation than those in the girder so as to avoid interference when the steel is being placed. Another very troublesome point is the intersection of columns with beams and girders.

5. Make certain that hooked and bent bars can be placed and have adequate concrete protection.

6. All bars, straight or bent, requiring hooks should be so designated by the designer.

7. Length of laps, points of bend, and extension of bars should be specified by the designer. Do not use the ratios L/7, L/5, and L/4 shown on Typical Drawings unless justified by structural analysis.

8. Be sure that unusual bends can be made with standard bending equipment.

9. Avoid ordering accessories such as bolsters and high-chairs in 1/8-inch increments of heights, as stock sizes come in 1/4-inch increments.

10. It is advisable to prepare placing drawings the same size as the engineering drawings.

11. For special or unusual conditions, be sure that adequate details are shown for proper placing of the reinforcement as the average steel setter does not understand engineering principles. Examples are cantilevers and continuous footings in which the reinforcement is in the opposite side from which the steel setter is accustomed.

12. Schedule dowels with the footings rather than in the column schedule.

13. Do not forget to splice column bars at top of upturned beams rather than at floor level.

14. Do not detail bar lengths to fractions of an inch. See Section 203 for standard tolerances. Increments of 3 inches for straight bars are recommended.

15. Where the lengths are not specifically fixed, such as for temperature steel, slabs on ground, and walls, use stock lengths or lengths which can be cut from stock lengths with a minimum of waste.

16. When a member has a break in its direction so that the reinforcement in tension tends to separate from the body of the concrete, special anchorage must be provided and shown in detail. Examples are the junction of stairs and landing, and where the soffit of a beam forms an angle.

17. Be sure that bent bars are not so large and unwieldy that they cannot be transported."

4.4.7 Fabricating Shop Practice. It is important that the designer and detailer are acquainted with practices of the fabricating shop, so the fabricating may be kept as simple and economical as possible. We are therefore quoting the following articles from the Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI Standard 315-48).

"202. Warehouse Stock

"There is some variation in the stock carried in different localities and also by fabricators in the same locality. The information which follows should be supplemented by information from local fabricators.

"Fabricators ordinarily stock only one or two grades of reinforcing steel. For bars, intermediate grade billet steel and rail steel are by far the most common, although other grades may at times be available. For spirals, hot rolled rods of intermediate grade are commonly stocked, although other grades and cold drawn wire may be available."

The following table of standard sizes of reinforcing bars is from ASTM Specification A305.

Bar No.	Unit Wt Lbs/Ft	Nominal Dimensions Round Sections		
		Diameter-Inches Decimal	Cross-Sectional Area Sq In.	Perimeter
2	0.167	0.250	0.05	0.785
3	0.376	0.375	0.11	1.178
4	0.668	0.500	0.20	1.571
5	1.043	0.625	0.31	1.963
6	1.502	0.750	0.44	2.356
7	2.044	0.875	0.60	2.749
8	2.670	1.000	0.79	3.142
9	3.400	1.128	1.00	3.544
10	4.303	1.270	1.27	3.990
11	5.313	1.410	1.56	4.430

Bar numbers are based on the number of 1/8 inches in the nominal diameter of the section.

Bar No. 2 in plain rounds only.

Bars numbered 9, 10, and 11 correspond to former 1" square, 1-1/8" square, and 1-1/4" square sizes and are equivalent to those former standard bar sizes in weights and nominal cross-sectional areas.

The quotation from ACI Standard 315-48 is continued below.

"The quality of reinforcing steel is governed by specifications of the American Society for Testing Materials, designated as follows:

"Billet-Steel Bars for Concrete Reinforcement,
Designation A-15.
Rail-Steel Bars for Concrete Reinforcement,
Designation A-16.
Axle-Steel Bars for Concrete Reinforcement,
Designation A-160.
Cold-Drawn Steel Wire for Concrete Reinforcement,
Designation A-82.
Welded-Steel Wire Fabric for Concrete Reinforcement,
Designation A-185.

"With the exception of the No. 2, all of these sizes are ordinarily furnished as deformed bars, the type of deformation depending upon the rolling mill. The No. 2 bar is generally available only as a plain bar.

"Bars are stocked by most fabricators in lengths of sixty feet. By special arrangement longer bars can sometimes be obtained but shipping limitations must be considered. Usually No. 2 bars are stocked in twenty-foot lengths because of the difficulty of handling such light bars in greater lengths.

"203. Tolerances

"Practical limitations of equipment and speed of production make it necessary to establish certain tolerances in fabrication which can be met with standard shop equipment. Where greater accuracy is required than given by the tolerances which follow, it must be definitely stated, as an extra charge is made for this type of fabrication.

"Reinforcing bars are cut to length by shearing with a tolerance of one inch more or one inch less than the specified length. Where more exact lengths or finished ends are required, the bars must be cut by special shears, or by attachments to regular shears, or by cold sawing.

"The dimensions of a bent bar are measured out to out of bar. The tolerance for the over-all bent length is one-half inch more or one-half inch less than the specified dimension for No. 7 bars or smaller, and one inch more or one inch less for larger bars. For a truss bar the tolerance in height is one-half inch less than the specified dimension. No greater height is permitted so that there is assurance that the bars can be placed within the depth of slab or beam specified.

"The diameter of column spirals is measured to the outside of the spiral. The tolerance is one-half inch more or one-half inch less than the specified diameter."

5. Structural Steel

5.1 Use and Exposure. Structural steel is not used extensively in the work of the Soil Conservation Service, and hence this discussion is general and brief. Structural shapes have limited use in the construction of (1) small I-beam bridges, (2) light trusses for supporting irrigation flumes and pipes, (3) interlocking sheet steel piling drop spillways, (4) bents for supporting flumes, pipes, and suspension span cables, (5) trash racks, (6) drainage, irrigation, and flood control gates, and (7) other structural accessories such as ladders, foot bridges, etc.

Such varied use provides wide differences in exposure conditions and probable corrosion hazard. In most cases the steel will be unprotected from the natural forces of wind, rain, sleet, snow, etc.; and in some cases it will be in direct contact with the earth and subject to alternate wet and dry conditions.

5.2 Design. The latest edition of the "Standard Specifications for Highway Bridges" adopted by The American Association of State Highway Officials should be used as the basic reference for unit stresses and design criteria insofar as they are applicable to the problem under study. Section 4, page 144 of the fifth edition (1949) of the above reference presents allowable unit stresses and Section 6, page 166, presents design criteria. These unit stresses should be used where exposure conditions are comparable to those normally encountered in highway bridges. For more severe exposure conditions it is recommended that the basic allowable unit stresses be reduced 10 percent.

The principles, procedure, and formulas required in the design of the relatively simple structures encountered in our work are readily available in many texts and handbooks, and it seems unnecessary to duplicate them here.

STRUCTURAL DESIGN:
STRUCTURAL TIMBER, UNIT WORKING STRESSES OF VARIOUS SPECIES FOR DIFFERENT EXPOSURE CONDITIONS

BENDING AND SHEARING STRESSES, IN POUNDS PER SQUARE INCH

Species of Timber	Transverse Bending Stresses										Horizontal Shearing Stress	
	All Sizes		Thickness, 5" and Over				Thickness, 4" and Under					
	Continuously Dry		Occasionally Wet		Usually Wet		Occasionally Wet		Usually Wet		Dry or Wet	
	Select	Common	Select	Common	Select	Common	Select	Common	Select	Common	Select	Common
Ash, White, commercial	1400	1120	1200	960	1000	800	1070	910	890	760	125	100
Beech and Birch, Yellow	1500	1200	1300	1040	1000	800	1150	980	890	760	125	100
Cedar, Port Orford and Alaska	1100	880	1000	800	900	720	890	760	800	680	90	72
Cedar, Western Red	900	720	800	640	750	600	710	600	670	570	80	64
Cedar, Northern and Southern White	750	600	650	520	600	480	580	490	530	450	70	56
Chestnut	950	760	850	680	700	560	760	650	620	530	90	72
Cypress, Southern	1300	1040	1100	880	900	720	980	830	800	680	100	80
Douglas Fir, Coast Region	1600	1200	1385	1040	1065	800	1235	985	950	755	90	72
Douglas Fir, Coast Region, Dense	1750	1400	1515	1215	1165	935	1350	1150	1035	880	105	84
Douglas Fir, Rocky Mountain	1100	880	900	720	700	560	800	680	620	530	85	68
Fir, White, commercial	1100	880	900	720	800	640	800	680	710	600	70	56
Fir, Balsam	900	720	750	600	600	480	670	570	530	450	70	56
Hemlock, Western	1300	1040	1100	880	900	720	980	830	800	680	75	60
Hemlock, Eastern	1100	880	900	720	800	640	800	680	710	600	70	56
Hickory	1900	1520	1500	1200	1200	960	1330	1130	1070	910	140	112
Larch, Western	1200	960	1100	880	900	720	980	830	800	680	100	80
Maple, Sugar and Black	1500	1200	1300	1040	1000	800	1150	980	890	760	125	100
Maple, Red and Silver	1000	800	900	720	700	560	800	680	620	530	100	80
Oak, Red and White, commercial	1400	1120	1200	960	1000	800	1070	910	890	760	125	100
Pine, Southern Yellow	....	1200	....	1040	....	800	....	985	...	755	...	88
Pine, Southern Yellow, Dense	1750	1400	1515	1215	1165	935	1350	1145	1035	880	128	103
Pine, White, Yellow, Sugar, Ponderosa	900	720	800	640	750	600	710	600	670	570	85	68
Pine, Norway	1100	880	1000	800	800	640	890	760	710	600	85	68
Poplar, Yellow	1000	800	900	720	800	640	800	680	710	600	80	64
Redwood	1200	960	1000	800	800	640	890	760	710	600	70	56
Spruce, Red, White and Sitka	1100	880	900	720	800	640	800	680	710	600	85	68
Spruce, Engelmann	750	600	650	520	500	400	580	490	440	370	70	56
Tamarack	1200	960	1100	880	900	720	980	830	800	680	95	76

COMPRESSIVE STRESSES AND ELASTIC MODULI, IN POUNDS PER SQUARE INCH

Species of Timber	Compressive Stresses									Average Modulus of Elasticity
	Perpendicular to Grain			Parallel with Grain						
	Continuously Dry	Occasionally Wet	Usually Wet	Continuously Dry		Occasionally Wet		Usually Wet		
	Select or Common			Select	Common	Select	Common	Select	Common	All Classes
Ash, White, commercial	500	375	300	1100	880	1000	800	900	720	1,500,000
Beech and Birch, Yellow	500	375	300	1200	960	1100	880	900	720	1,600,000
Cedar, Alaska	250	200	150	800	640	750	600	650	520	1,200,000
Cedar, Port Orford	250	200	150	900	720	825	660	750	600	1,200,000
Cedar, Western Red	200	150	125	700	560	700	560	650	520	1,000,000
Cedar, Northern and Southern White	175	140	100	550	440	500	400	450	360	800,000
Chestnut	300	200	150	800	640	700	560	600	480	1,000,000
Cypress, Southern	300	225	200	1100	880	1000	800	800	640	1,200,000
Douglas Fir, Coast Region	345	240	215	1175	880	1065	800	905	680	1,600,000
Douglas Fir, Coast Region, Dense	380	260	235	1285	1025	1165	935	990	795	1,600,000
Douglas Fir, Rocky Mountain	275	225	200	800	640	800	640	700	560	1,200,000
Fir, White, commercial	300	225	200	700	560	700	560	600	480	1,100,000
Fir, Balsam	150	125	100	700	560	600	480	500	400	1,000,000
Hemlock, Western	300	225	200	900	720	900	720	800	640	1,400,000
Hemlock, Eastern	300	225	200	700	560	700	560	600	480	1,100,000
Hickory	600	400	350	1500	1200	1200	960	1000	800	1,800,000
Larch, Western	325	225	200	1100	880	1000	800	800	640	1,300,000
Maple, Sugar and Black	500	375	300	1200	960	1100	880	900	720	1,600,000
Maple, Red and Silver	350	250	200	800	640	700	560	600	480	1,100,000
Oak, Red and White, Commercial	500	375	300	1000	800	900	720	800	640	1,500,000
Pine, Southern Yellow	...	...	...	....	880	....	800	...	680	1,600,000
Pine, Southern Yellow, Dense	380	260	235	1285	1025	1165	935	990	795	1,600,000
Pine, White, Yellow, Sugar and Ponderosa	250	150	125	750	600	750	600	650	520	1,000,000
Pine, Norway	300	175	150	800	640	800	640	700	560	1,200,000
Poplar, Yellow	250	150	125	800	640	700	560	600	480	1,100,000
Redwood	250	150	125	1000	800	900	720	750	600	1,200,000
Spruce, Red, White and Sitka	250	150	125	800	640	750	600	650	520	1,200,000
Spruce, Engelmann	175	140	100	600	480	550	440	450	360	800,000
Tamarack	300	225	200	1000	800	900	720	800	640	1,300,000

REFERENCE

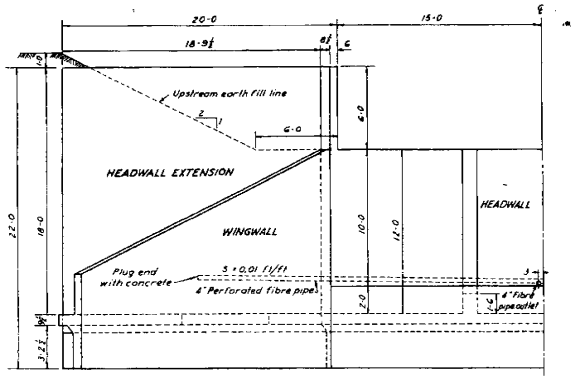
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
H. H. Bennett, Chief

ENGINEERING STANDARDS UNIT

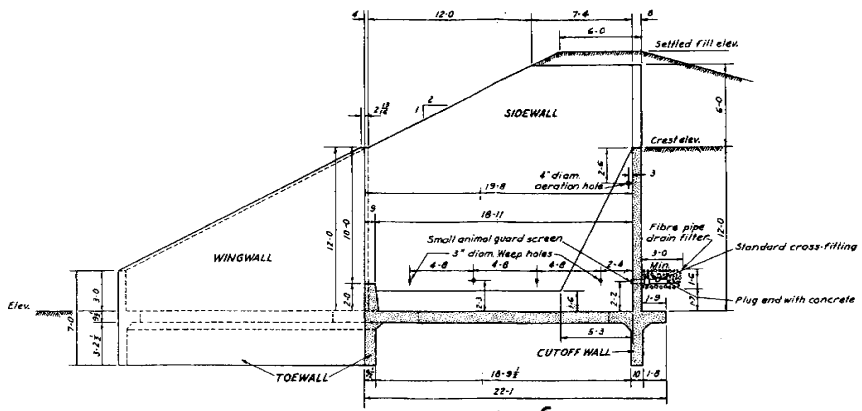
STANDARD DWG. NO.
ES - 26
SHEET 1 OF 1
DATE 5-4-50

shown, but the drawing should not be cluttered up with unnecessary views, marks, and notes. The first sheet of this drawing will be the layout sheet which will show the over-all structure features and dimensions. This sheet will usually contain a bill of materials and lumber cutting bill. The following detail sheets will show the detailed construction information, such as details of members, joints, etc. If the structure is complex and has numerous detail sheets, it is best to show a bill of material and lumber cutting bill on each detail sheet with a summary shown on the layout sheet. The allowable unit stresses used in the design should be indicated on the layout sheet along with whether the design is based on standard dressed, standard rough, or special sizes of lumber.

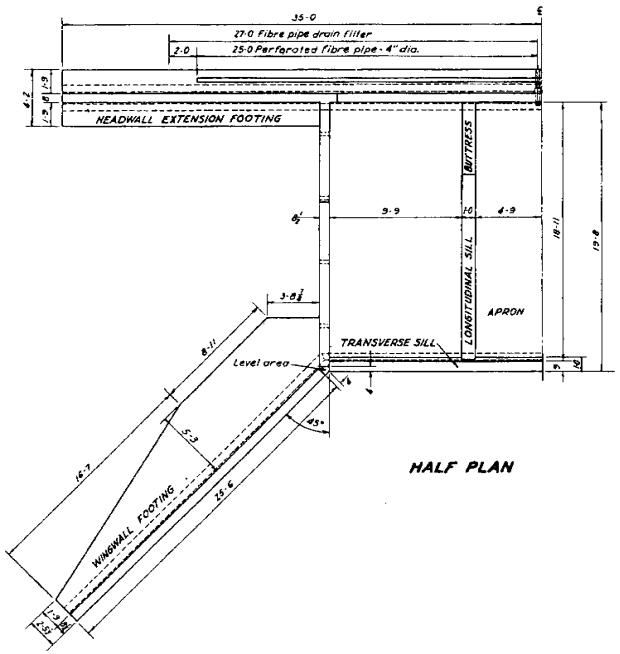
*U.S. GOVERNMENT PRINTING OFFICE: 1981-0-727-744/1701



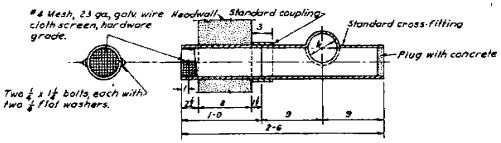
HALF DOWNSTREAM ELEVATION



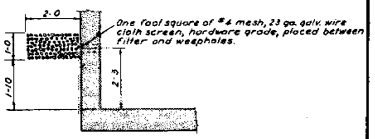
CROSS-SECTION ALONG ℄



HALF PLAN



DETAIL OF SMALL ANIMAL GUARD SCREEN AND OUTLET END OF 4" FIBRE PIPE



SECTION OF WEEP HOLE DRAIN FILTER

NOTES:

The structure is symmetrical about its centerline except the 4" fibre pipe and drain filter which are symmetrical about the outlet of the fibre pipe.
The fibre pipe shall be Orangeburg fibre pipe or equal.
The fibre pipe drain filter shall be a clean pit run sand and gravel mixture and shall be continuous along the pipe and extend 2'-0" beyond the ends of the pipe.
The weep hole drain filter shall be a clean pit-run sand and gravel mixture and shall extend the length of the sidewall.
Chamfer all exposed edges ¾"
All fillets 6"
The longitudinal and transverse sills shall be poured monolithically with the apron and without the use of a horizontal construction joint.
All reinforcing steel shall have at least 2" of concrete cover and in faces poured against earth, it shall have at least 3" of concrete cover.
All reinforcing bar lengths shall be in increments of 3"

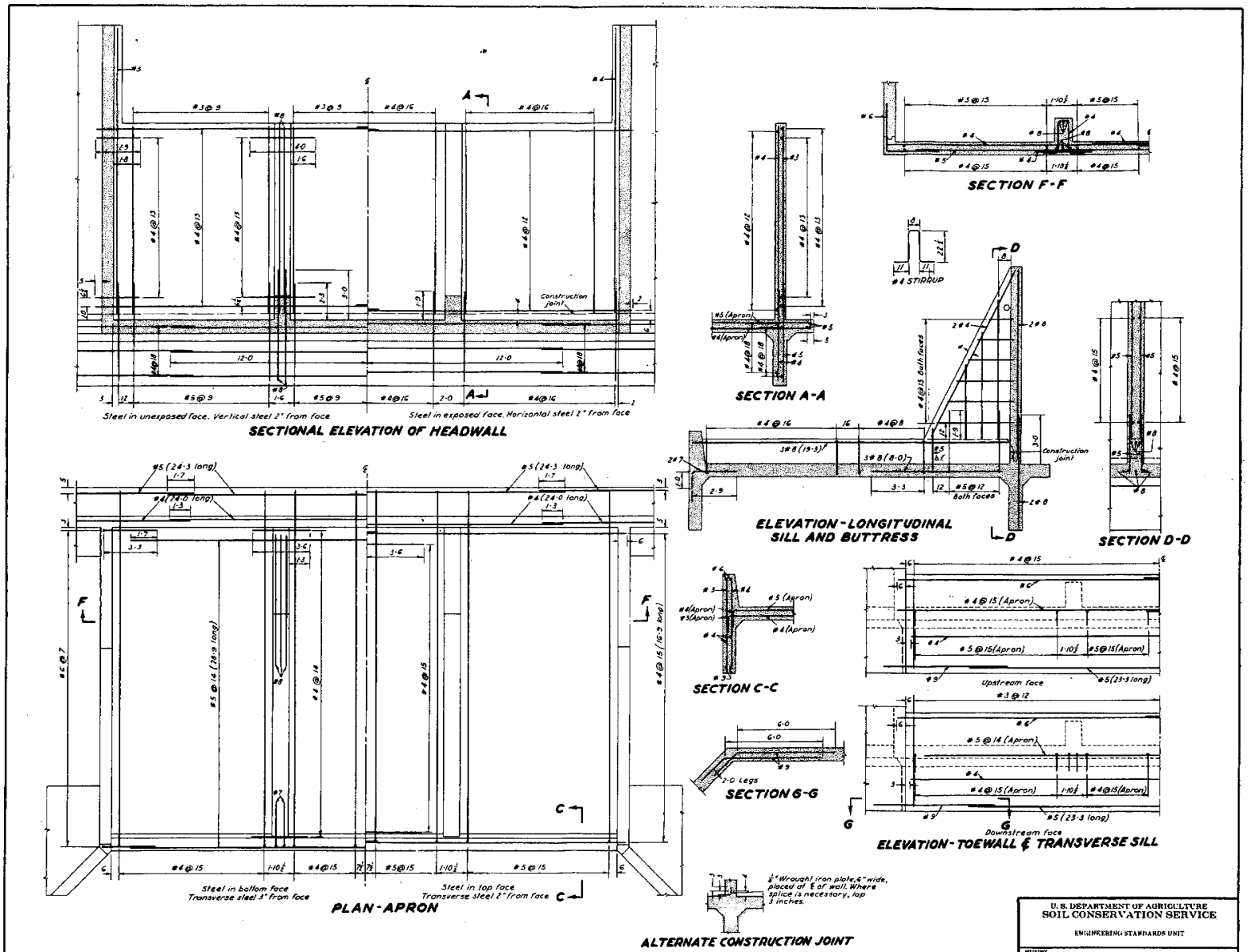
ENGINEERING DRAWING

R/C Drop Spillway: F = 10'-0, h = 6'-0, L = 30'-0

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING STANDARDS UNIT

REFERENCE

ENGINEERING APPROVAL M. M. Culp	CARTOGRAPHIC APPROVAL J. J. Carey
DESIGNED BY C.A. Reese	DRAWN BY R. Bayeson
CHECKED BY J.R. Brownell	STANDARD DWG. NO. ES-2108-30 BE
DATE 1-13-51	SHEET 1 OF 3 SHEETS



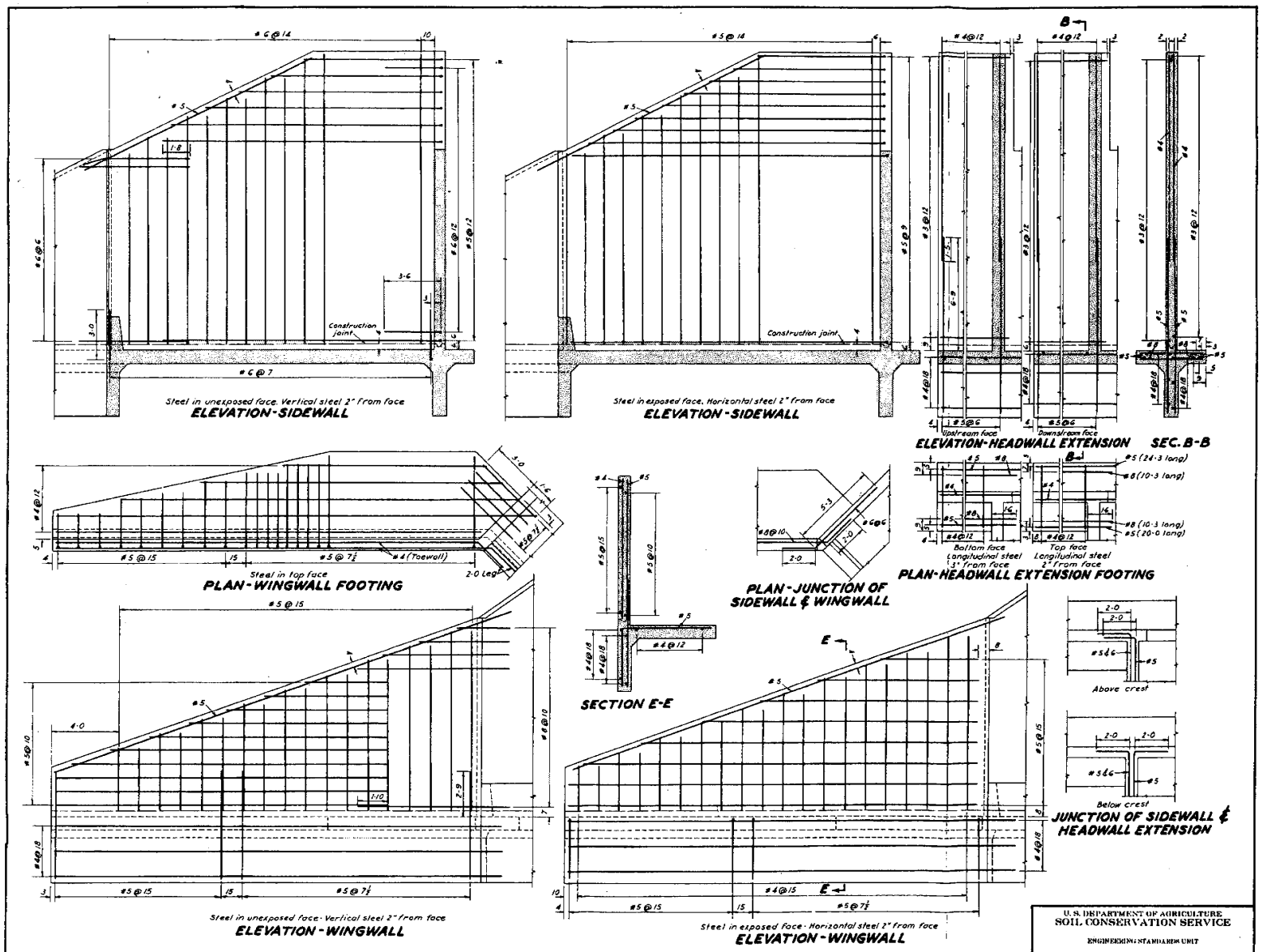
ENGINEERING DRAWING

R/C Drop Spillway - F=10-0, h=6-0, L=30-0

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING STANDARDS UNIT

REFERENCE

ENGINEERING APPROVAL M. M. Culp	CARTOGRAPHIC APPROVAL J. T. Carey
DESIGNED BY C.A. Reese	DRAWN BY R. Boyeson
CHECKED BY J.R. Brownell	STANDARD DWG. NO. ES-2108-30 BE
DATE 1-13-51	SHEET 2 OF 3 SHEETS



ENGINEERING DRAWING

D/C Drop Spillway F=10.0, h=6.0, L=30.0

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING STANDARDS UNIT

REFERENCE	
ENGINEERING APPROVAL M. M. Culp	CARTOGRAPHIC APPROVAL J. T. Casey
DESIGNED BY C.A. Reese	DRAWN BY R. Boyeson
CHECKED BY J.R. Brownell	STANDARD DWG. NO. ES-2108-30 BE
DATE 1-13-51	SHEET 3 OF 3 SHEETS

STEEL SCHEDULE

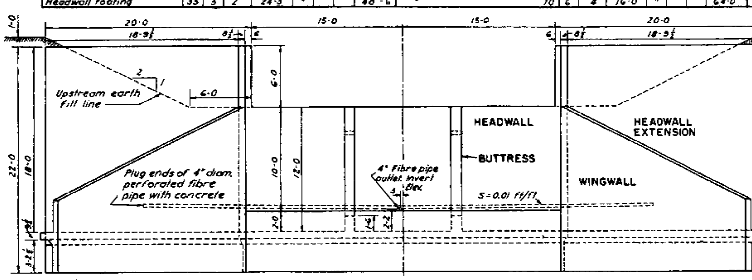
LOCATION	MARK	SIZE	QUAN.	LENGTH	TYPE	A	B	TOTAL FT.
Toewall	1	3	4	8-0	16	2-0	6-0	32-0
"	2	5	2	23-3	St			46-6
"	3	4	2	30-6	"			61-0
Toewall and transverse sill	4	3	31	5-6	"			170-6
" " " "	5	4	25	5-6	"			137-6
Toewall and wingwall	6	5	46	6-6	"			299-0
" " "	7	5	18	6-6	"			117-0
Wingwall footing	8	4	2	25-6	"			51-0
" "	9	4	2	29-0	"			58-0
" "	10	4	2	23-9	"			47-6
" "	11	4	2	18-3	"			36-6
" "	12	4	2	12-3	"			24-6
Wingwall footing and apron	13	5	8	4-6	"			36-0
Toewall and wingwall footing	14	5	30	9-0	2	3-6	5-6	270-0
" " " "	15	5	8	8-6	"	3-6	5-0	68-0
" " " "	16	5	8	8-0	"	3-6	4-6	64-0
" " " "	17	5	4	7-6	"	3-6	4-0	30-0
" " " "	18	5	4	6-9	"	3-6	3-3	27-0
" " " "	19	5	4	6-6	"	3-6	3-0	26-0
" " " "	20	5	6	5-6	"	3-6	2-0	33-0
Toewall under wingwall	21	4	6	27-6	16	25-6	2-0	165-0
" " "	22	4	6	28-0	"	26-0	2-0	168-0
Cutoff wall	23	4	18	24-0	St			432-0
Cutoff wall and headwall	24	5	39	6-0	"			234-0
" " " "	25	8	4	6-9	"			27-0
Cutoff wall and hdwl. ext.	26	5	156	10-6	"			1638-0
Cutoff wall and headwall	27	4	23	5-6	"			126-6
Apron and sidewall	28	6	68	6-3	2	3-3	3-0	425-0
Apron	29	5	17	28-9	St			488-9
"	30	4	34	3-6	"			119-0
Apron and toewall	31	4	24	22-9	2	21-9	1-0	546-0
Longitudinal sill	32	4	28	6-3	S4	See detail		175-0
Longitudinal sill and toewall	33	7	4	3-9	2	2-9	1-0	15-0
Longitudinal sill	34	8	6	8-0	St			48-0
Headwall footing	35	3	2	24-3	"			48-6

LOCATION	MARK	SIZE	QUAN.	LENGTH	TYPE	A	B	TOTAL FT.
Headwall extension footing	36	4	76	3-9	St			285-0
" " "	37	5	4	20-0	"			80-0
" " "	38	8	8	10-3	"			82-0
" " "	39	9	4	24-3	"			97-0
Apron and toewall	40	5	24	22-3	2	21-3	1-0	534-0
Apron	41	4	32	16-9	St			536-0
"	42	4	15	7-0	"			105-0
Longitudinal sill	43	8	6	19-3	"			115-6
Transverse sill	44	6	4	17-0	"			68-0
Buttress	45	5	16	3-9	"			60-0
"	46	5	4	2-9	"			11-0
Wingwall	47	5	4	10-9	"			43-0
"	48	5	4	10-0	"			40-0
"	49	5	4	9-0	"			36-0
"	50	5	4	8-3	"			33-0
"	51	5	4	7-3	"			29-0
"	52	5	4	6-6	"			26-0
"	53	5	4	5-6	"			22-0
"	54	5	4	4-9	"			19-0
"	55	5	4	3-9	"			15-0
Wingwall and sidewall	56	8	22	9-9	16	7-9	2-0	214-6
" " "	57	8	2	8-3	"	6-3	2-0	16-6
" " "	58	8	2	6-0	"	4-0	2-0	12-0
" " "	59	8	2	3-9	"	1-9	2-0	7-6
Wingwall	60	5	6	20-0	St			120-0
"	61	5	2	19-0	"			38-0
"	62	5	2	16-9	"			33-6
"	63	5	2	14-3	"			28-6
"	64	5	2	12-0	"			24-0
"	65	5	2	9-9	"			19-6
"	66	5	2	7-6	"			15-0
"	67	5	2	5-0	"			10-0
Sidewall	68	6	12	17-6	"			210-0
"	69	6	2	17-3	"			34-6
"	70	6	4	16-0	"			64-0

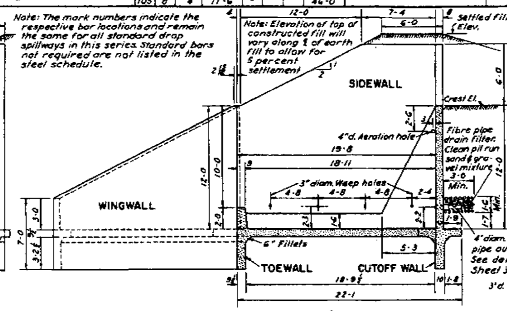
LOCATION	MARK	SIZE	QUAN.	LENGTH	TYPE	A	B	TOTAL FT.
Sidewall	71	6	4	14-9	St			59-0
"	72	6	4	13-9	"			55-0
"	73	6	4	12-6	"			50-0
"	74	6	4	11-6	"			46-0
Sidewall and headwall ext.	75	5	26	18-9	2	16-9	2-0	487-6
" " " "	76	5	2	18-3	2	16-3	2-0	36-6
" " " "	77	5	2	16-3	2	14-3	2-0	32-6
" " " "	78	5	2	14-3	2	12-3	2-0	28-6
" " " "	79	5	2	12-3	2	10-3	2-0	24-6
" " " "	80	5	2	10-3	2	8-3	2-0	20-6
" " " "	81	6	34	5-6	2	3-6	2-0	187-0
Sidewall and wingwall	82	6	46	7-3	16	5-3	2-0	333-6
Wingwall	83	4	4	10-6	St			42-0
"	84	4	4	9-6	"			38-0
"	85	4	4	8-9	"			35-0
"	86	4	4	8-0	"			32-0
"	87	4	4	7-0	"			28-0
"	88	4	4	6-0	"			24-0
"	89	4	4	5-3	"			21-0
"	90	4	4	4-6	"			18-0
"	91	4	4	3-6	"			14-0
"	92	4	4	2-9	"			11-0
"	93	5	4	25-0	"			100-0
"	94	5	2	24-0	"			48-0
"	95	5	2	20-6	"			41-0
"	96	5	2	16-9	"			33-6
"	97	5	2	13-3	"			26-6
"	98	5	2	9-9	"			19-6
"	99	5	2	6-0	"			12-0
Wingwall and sidewall	100	5	4	28-3	16	26-3	2-0	113-0
Headwall extension	101	4	80	12-6	St			1000-0
" "	102	3	72	19-6	"			1404-0
Headwall	103	3	2	17-6	"			35-0
"	104	3	37	11-6	"			425-6
"	105	8	4	11-6	"			46-0

LOCATION	MARK	SIZE	QUAN.	LENGTH	TYPE	A	B	TOTAL FT.
Headwall	106	4	22	17-3	St			379-6
"	107	4	20	2-9	"			55-0
"	108	8	20	4-0	"			80-0
Sidewall	109	5	14	17-6	"			245-0
"	110	5	4	16-0	"			64-0
"	111	5	4	15-0	"			60-0
"	112	5	4	13-9	"			55-0
"	113	5	4	12-6	"			50-0
"	114	5	4	11-6	"			46-0
Sidewall and headwall	115	5	32	21-3	2	19-3	2-0	680-0
Sidewall and headwall ext.	116	5	2	20-0	2	18-0	2-0	40-0
" " " "	117	5	2	18-6	2	16-6	2-0	37-0
" " " "	118	5	2	17-3	2	15-3	2-0	34-6
" " " "	119	5	2	15-6	2	13-6	2-0	31-0
" " " "	120	5	2	14-3	2	12-3	2-0	28-6
" " " "	121	5	2	12-9	2	10-9	2-0	25-6
" " " "	122	5	2	11-3	2	9-3	2-0	22-6
" " " "	123	5	2	9-9	2	7-9	2-0	19-6
Sidewall and wingwall	124	5	4	15-6	16	13-6	2-0	62-0
Headwall	125	4	2	17-6	St			35-0
"	126	4	21	11-6	"			241-6
"	127	4	24	17-0	"			408-0
Buttress	128	5	4	8-6	"			34-0
"	129	5	4	6-6	"			26-0
"	130	5	4	4-6	"			18-0
"	131	4	4	4-9	"			19-0
"	132	4	4	4-0	"			16-0
"	133	4	4	3-6	"			14-0
"	134	4	4	2-9	"			11-0
"	135	4	4	2-3	"			9-0
"	136	4	4	1-6	"			6-0
"	137	4	4	11-3	"			45-0

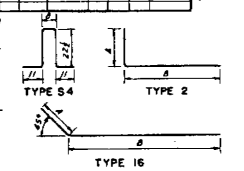
Note: The mark numbers indicate the respective bar locations and remain the same for all standard drop spillways in this series. Standard bars not required are not listed in the steel schedule.



DOWNSTREAM ELEVATION

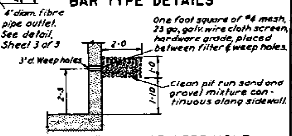


CROSS-SECTION ALONG ℄

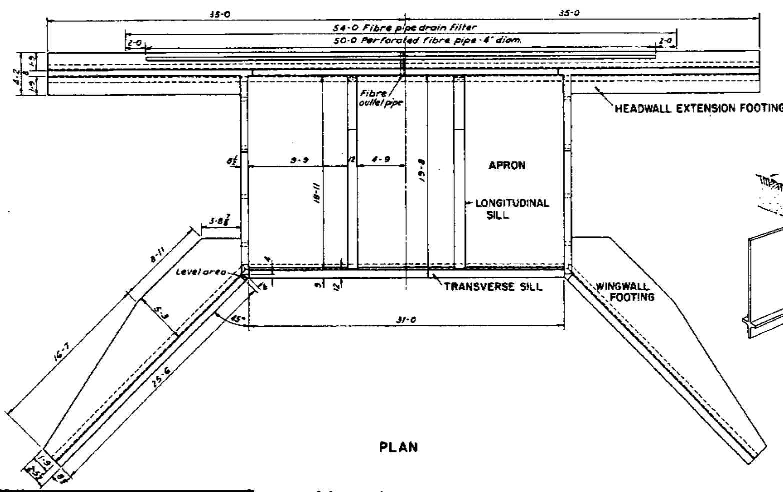


The bar schedule is listed in the approximate order of placement. Bars 1 through 46 are contained in the first pour. All reinforcing steel is round. "St" is an abbreviation for straight.

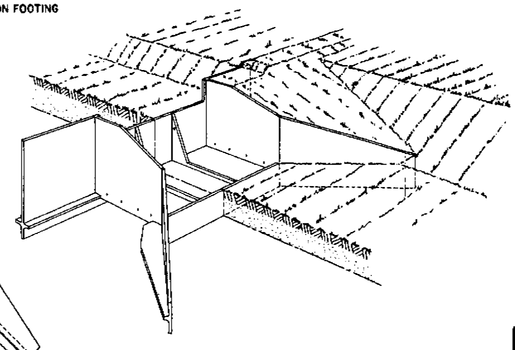
BAR TYPE DETAILS



SECTION OF WEEP HOLE AND DRAIN FILTER



PLAN



PERSPECTIVE VIEW

Not to scale.

QUANTITIES

Reinforced concrete		108.45 Cu. yds
Reinforcing steel		
# 3	2035.00 Ft.	765.16 Pounds
# 4	5655.50 "	3777.87 "
# 5	7361.25 "	7677.78 "
# 6	1532.00 "	2301.06 "
# 7	15.00 "	30.06 "
# 8	569.00 "	1519.23 "
# 9	32.00 "	108.80 "
Drainage-Perforated fibre pipe-4" d.		52.5 L.F.
Drainage-Drain filter		11.9 Cu. yd.

Class "B" Concrete
f_s = 20,000 lbs/sq. in.
f'_c = 3,000 "
f_c = 1,200 "

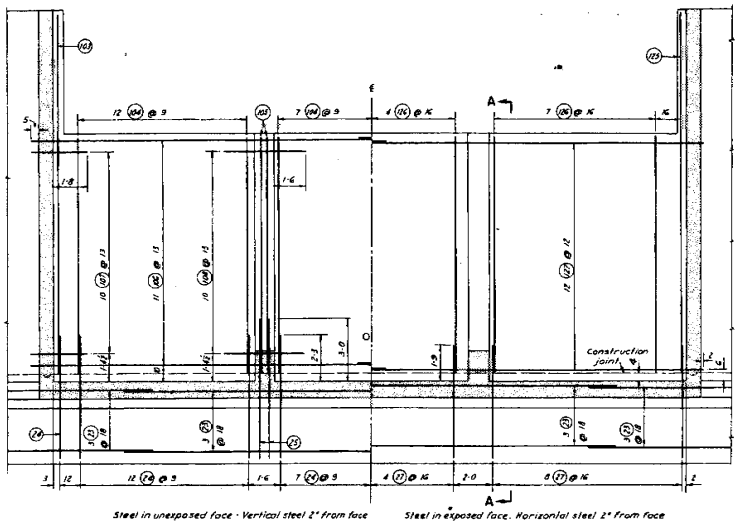
F = 10-0, h = 6-0, L = 30-0

Note: Structure is symmetrical about ℄ except for 4" fibre pipe drain and outlet.
Chamfer all exposed edges ¾".

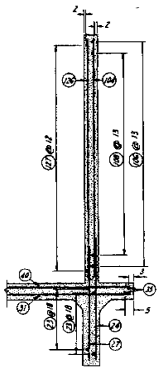
ENGINEERING STANDARDS UNIT
Engineering approval: M. M. Culp | Cartographic approval: [signature]
Designed by: C. A. Reese | Drawn by: R. Boyeson
Checked by: J. R. Brownell | Standard dwg. no. ES-2108-30B
Date: 2-5-51 | Sheet 1 of 3 sheets

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

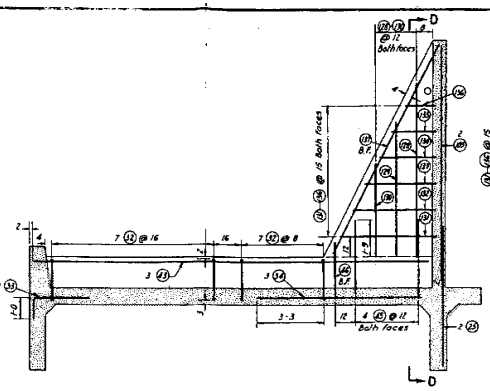
Reference | Engineering approval | Cartographic approval | Adapted by | Drawn by | Checked by | Drawing no. | Date | Sheet of sheets



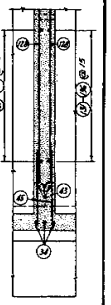
SECTIONAL ELEVATION OF HEADWALL



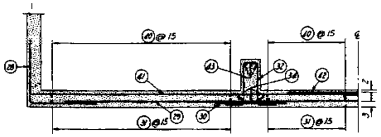
SECTION A-A



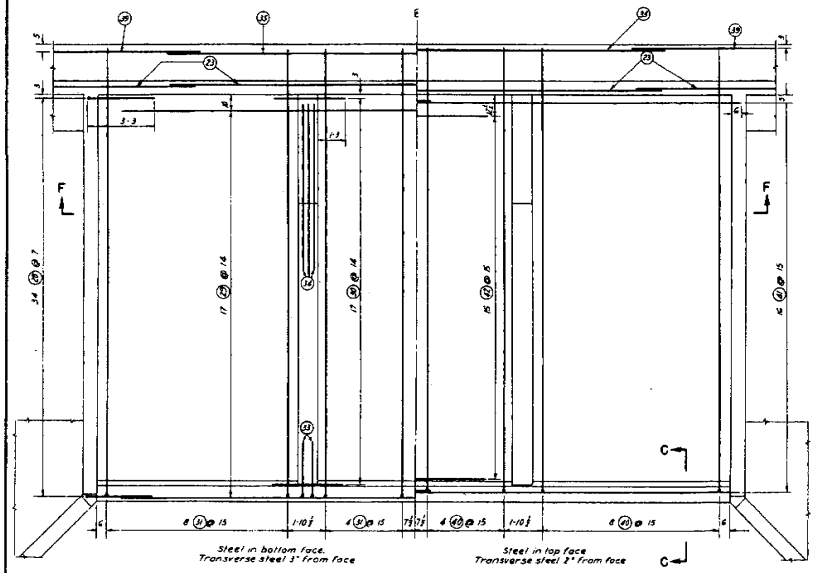
ELEVATION - LONGITUDINAL SILL AND BUTTRESS



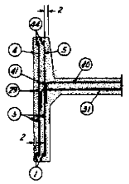
SEC. D-D



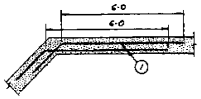
HALF - SECTION F-F



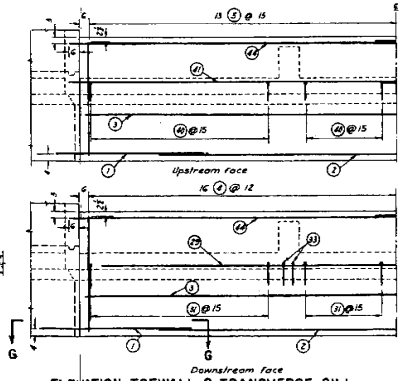
PLAN - APRON



SECTION C-C



SECTION G-G



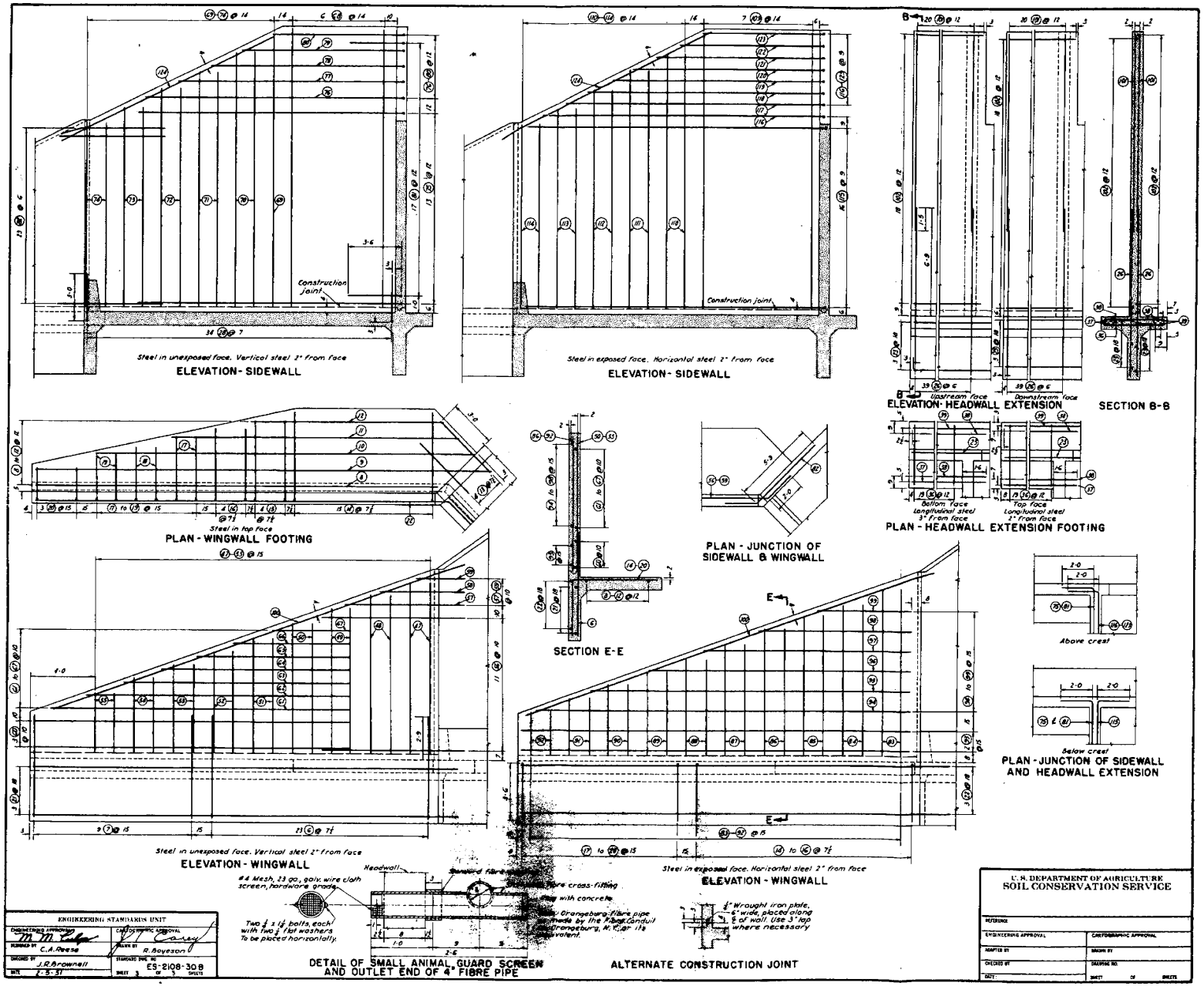
ELEVATION - TOEWALL & TRANSVERSE SILL

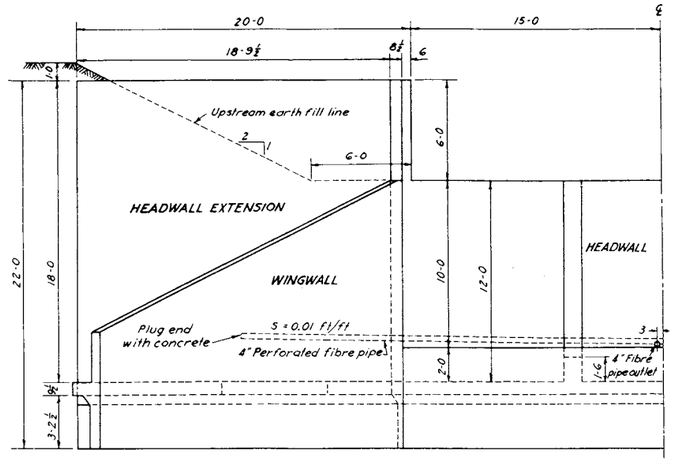
Note:
The longitudinal and transverse sills shall be poured monolithically with the apron and without use of a horizontal construction joint.
All dimensions showing distance from face of concrete to steel designate the clear distance.

ENGINEERING STANDARDS UNIT	
ENGINEERING APPROVAL M. M. Culp	CARTOGRAPHIC APPROVAL J. J. Casey
DESIGNED BY C. A. Reese	DRAWN BY R. Bowson
CHECKED BY J. R. Brownell	STANDARD DWG. NO. ES-2108-308
DATE 2-3-51	SHEET 2 OF 3 SHEETS

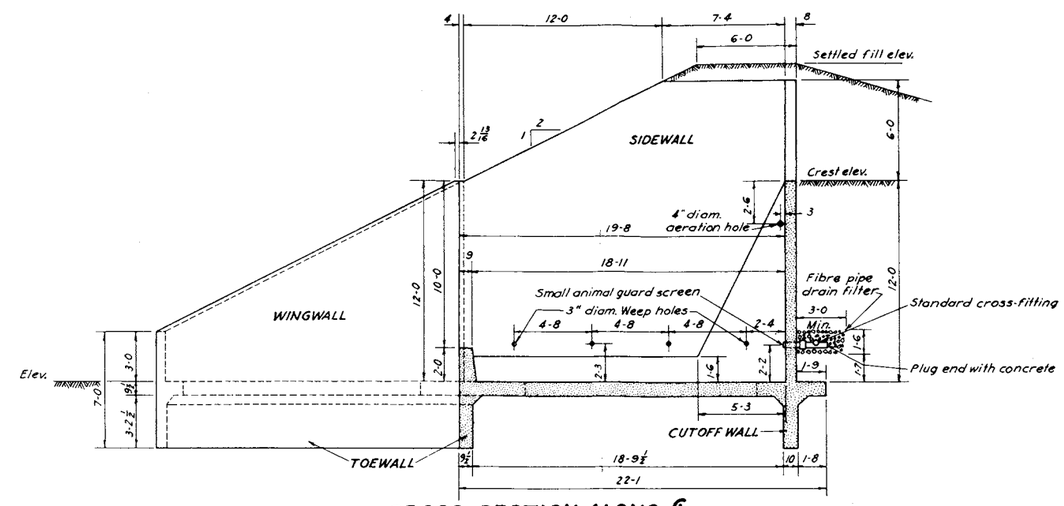
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

RESERVOIR	
ENGINEERING APPROVAL	CARTOGRAPHIC APPROVAL
ADAPTED BY	DRAWN BY
CHECKED BY	DRAWING NO.
DATE	SHEET OF SHEETS

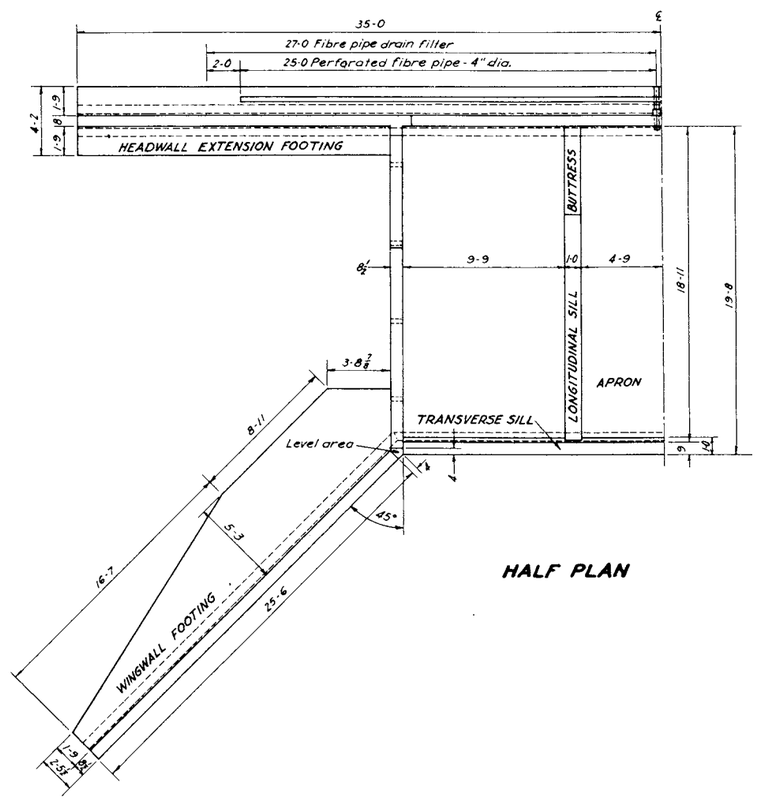

Steel in unexposed face. Vertical steel 2" from face
ELEVATION - SIDEWALL
Construction joint
Steel in exposed face. Horizontal steel 2" from face
ELEVATION - SIDEWALL
Construction joint
Upstream face
Downstream face
ELEVATION - HEADWALL EXTENSION
SECTION B-B
Steel in top face
PLAN - WINGWALL FOOTING
SECTION E-E
PLAN - JUNCTION OF SIDEWALL & WINGWALL
Bottom face
Longitudinal steel 3" from face
Top face
Longitudinal steel 2" from face
PLAN - HEADWALL EXTENSION FOOTING
Above crest
Below crest
PLAN - JUNCTION OF SIDEWALL AND HEADWALL EXTENSION
Steel in unexposed face. Vertical steel 2" from face
ELEVATION - WINGWALL
Steel in exposed face. Horizontal steel 2" from face
ELEVATION - WINGWALL
#4 Mesh, 23 ga., galv. wire cloth screen, hardware grade
Headwall
Two ½ x 1½ bolts each with two ½ flat washers To be placed horizontally.
DETAIL OF SMALL ANIMAL GUARD SCREEN AND OUTLET END OF 4" FIBRE PIPE
½" Wrought iron plate, 6" wide, placed along ½ of wall. Use 3" lap where necessary
ALTERNATE CONSTRUCTION JOINT
ENGINEERING STANDARDS UNIT
ENGINEERING APPROVAL
CARTOGRAPHIC APPROVAL
DRAWN BY R. Boyeson
CHECKED BY J.R. Brownell
DATE 2-5-51
STANDARD DWG. NO. ES-2108-308
SHEET 3 OF 3 SHEETS
U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
REVISIONS
ENGINEERING APPROVAL
CARTOGRAPHIC APPROVAL
ADAPTED BY
DRAWN BY
CHECKED BY
DRAWING NO.
DATE
SHEET OF SHEETS



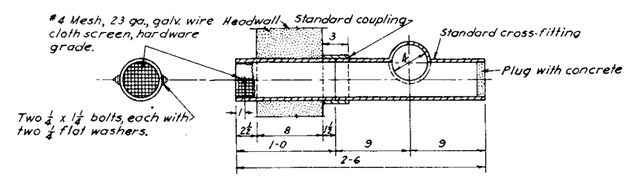
HALF DOWNSTREAM ELEVATION



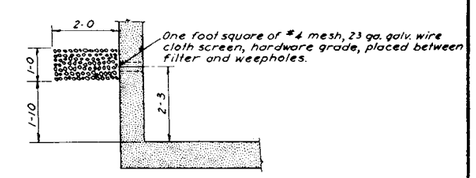
CROSS-SECTION ALONG ℄



HALF PLAN



DETAIL OF SMALL ANIMAL GUARD SCREEN AND OUTLET END OF 4" FIBRE PIPE



SECTION OF WEEP HOLE DRAIN FILTER

NOTES:

The structure is symmetrical about its centerline except the 4" fibre pipe and drain filter which are symmetrical about the outlet of the fibre pipe.

The fibre pipe shall be Orangeburg fibre pipe or equal.

The fibre pipe drain filter shall be a clean pit run sand and gravel mixture and shall be continuous along the pipe and extend 2'-0" beyond the ends of the pipe.

The weep hole drain filter shall be a clean pit-run sand and gravel mixture and shall extend the length of the sidewall.

Chamfer all exposed edges 3/4"

All fillets 6"

The longitudinal and transverse sills shall be poured monolithically with the apron and without the use of a horizontal construction joint.

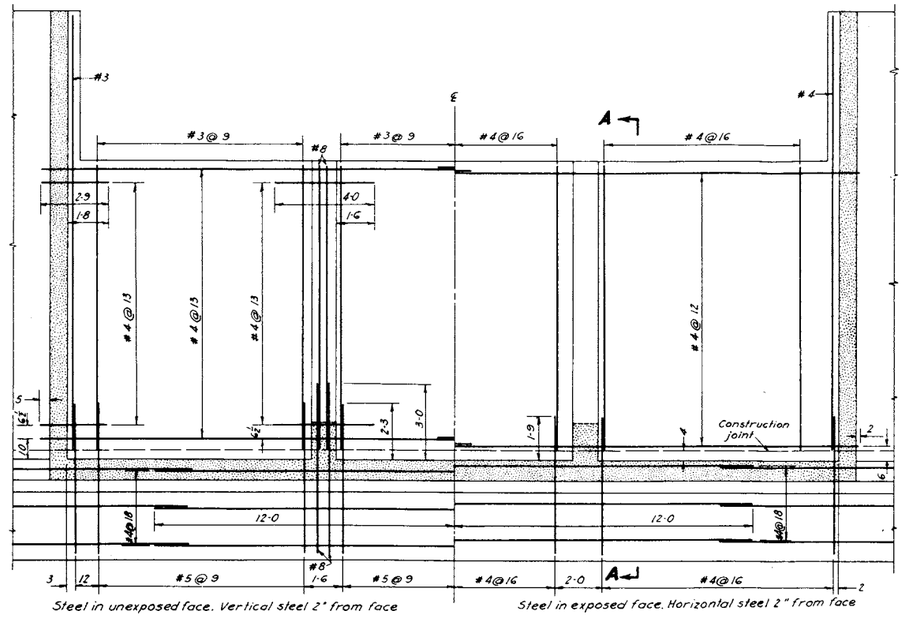
All reinforcing steel shall have at least 2" of concrete cover and in faces poured against earth, it shall have at least 3" of concrete cover.

All reinforcing bar lengths shall be in increments of 3"

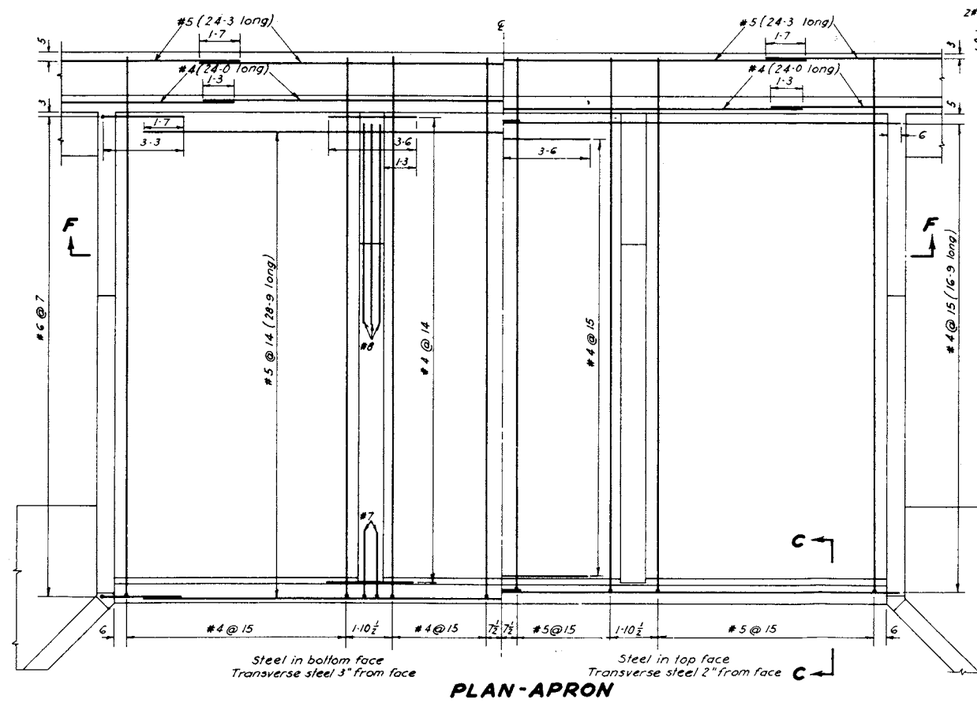
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING STANDARDS UNIT

REFERENCE	
ENGINEERING APPROVAL: W. M. Culp	CARTOGRAPHIC APPROVAL: V. Carey
DESIGNED BY: C.A. Reese	DRAWN BY: R. Boyeson
CHECKED BY: J.R. Brownell	STANDARD DWG. NO.: ES-2108-30 BE
DATE: 1-13-51	SHEET 1 OF 3 SHEETS

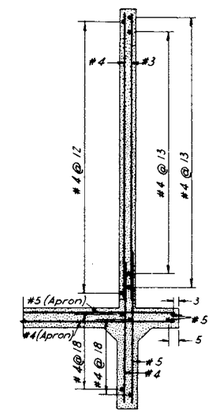
ENGINEERING DRAWING R/C Drop Spillway - F = 10'-0, h = 6'-0, L = 30'-0



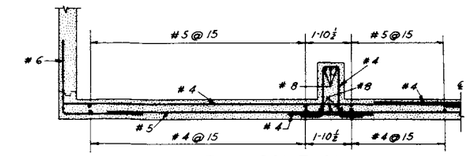
SECTIONAL ELEVATION OF HEADWALL



PLAN-APRON



SECTION A-A



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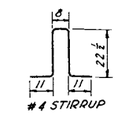
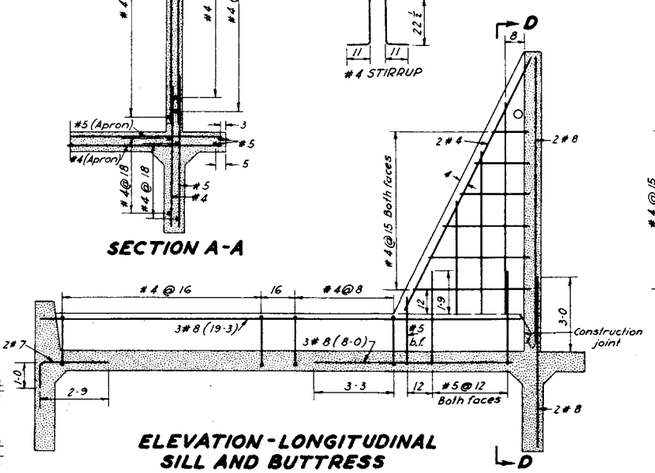
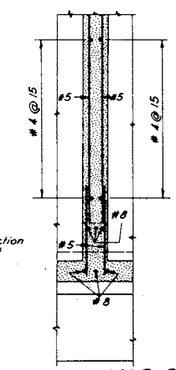


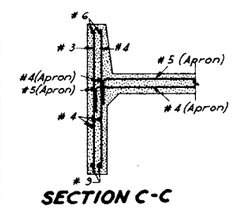
#4 STIRRUP



ELEVATION-LONGITUDINAL SILL AND BUTTRESS



SECTION D-D



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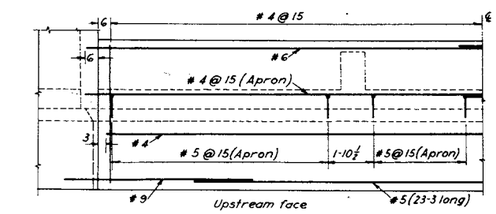

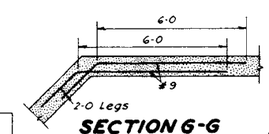

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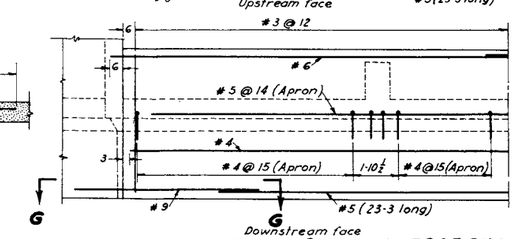
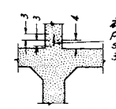


ELEVATION-TOEWALL & TRANSVERSE SILL



¼" Wrought iron plate, 6" wide, placed at ℄ of wall. Where splice is necessary, lap 3 inches.

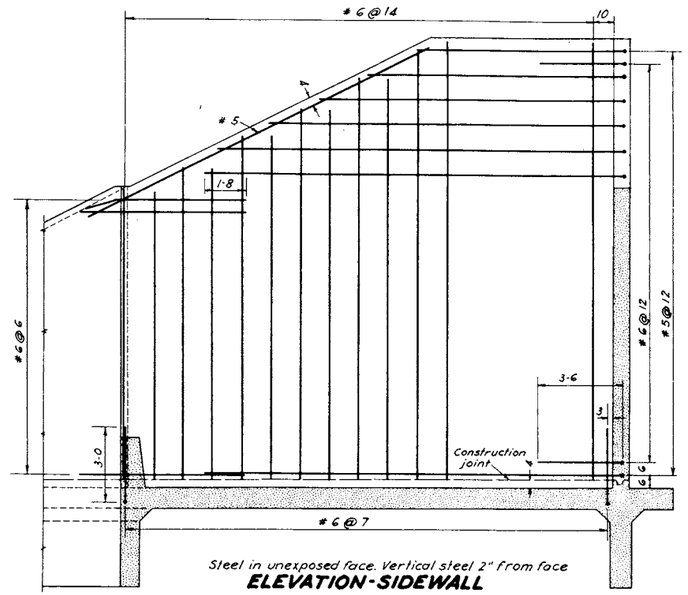
ALTERNATE CONSTRUCTION JOINT

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

ENGINEERING STANDARDS UNIT

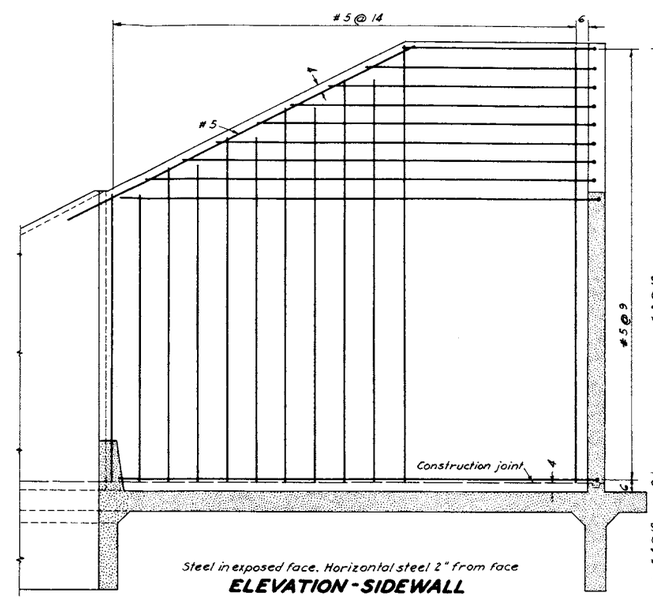
REFERENCE	
ENGINEERING APPROVAL M. M. Culp	CARTOGRAPHIC APPROVAL F. T. Carey
DESIGNED BY C.A. Reese	DRAWN BY R. Boyeson
CHECKED BY J.R. Brownell	STANDARD DWG. NO. ES-2108-30 BE
DATE 1-13-51	SHEET 2 OF 3 SHEETS

ENGINEERING DRAWING R/C Drop Spillway - F=10·0, h=6·0, L=30·0



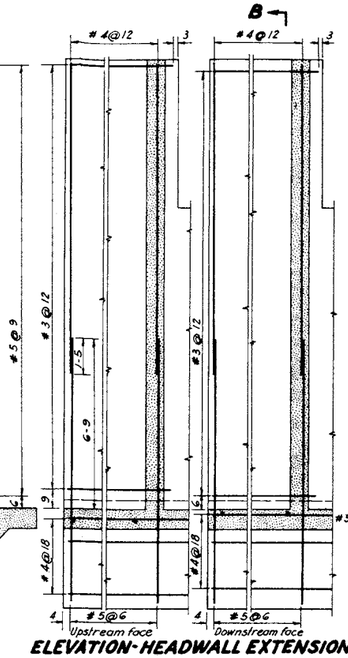
Steel in unexposed face. Vertical steel 2" from face

ELEVATION-SIDEWALL

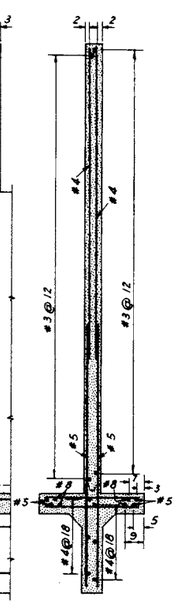


Steel in exposed face. Horizontal steel 2" from face

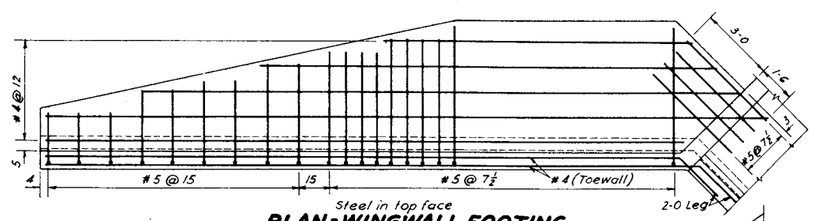
ELEVATION-SIDEWALL



ELEVATION-HEADWALL EXTENSION

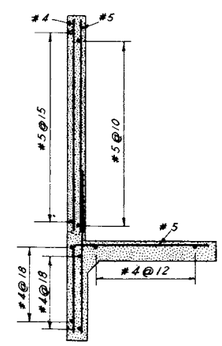


SEC. B-B

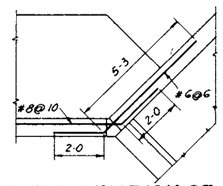


Steel in top face

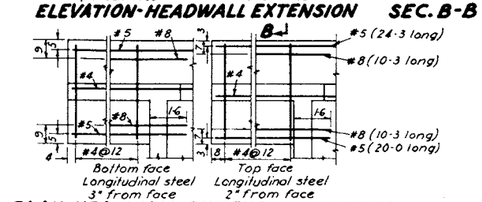
PLAN-WINGWALL FOOTING



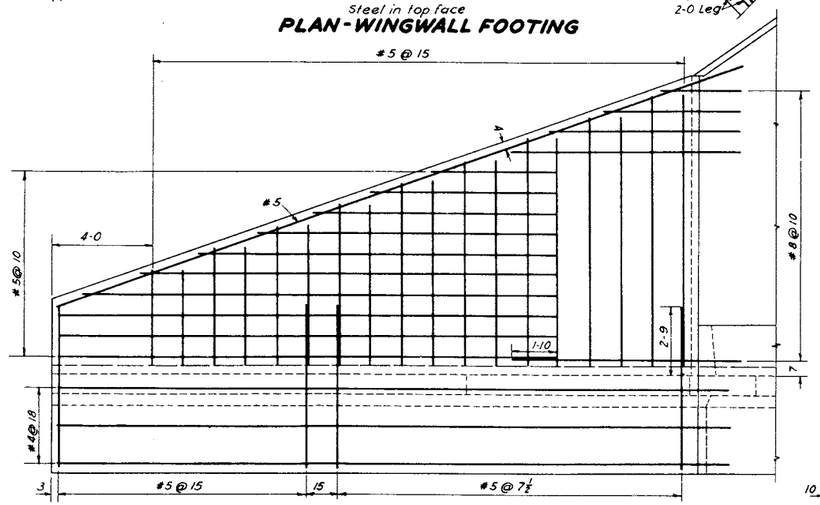
SECTION E-E



PLAN-JUNCTION OF SIDEWALL & WINGWALL

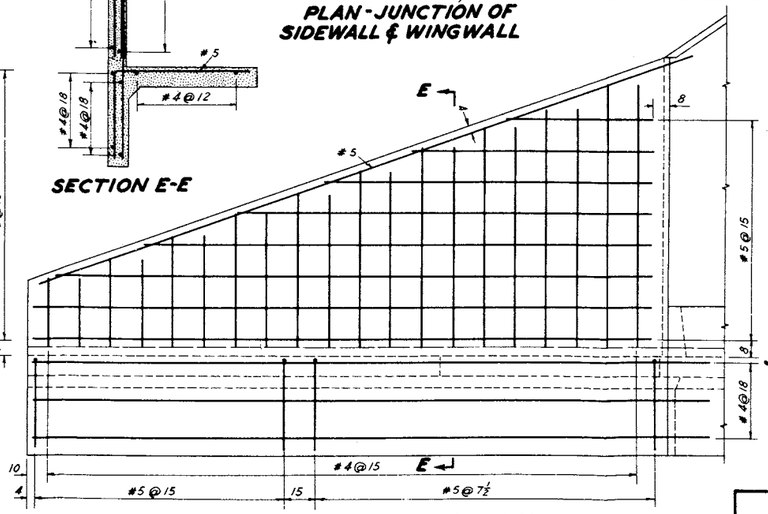


PLAN-HEADWALL EXTENSION FOOTING



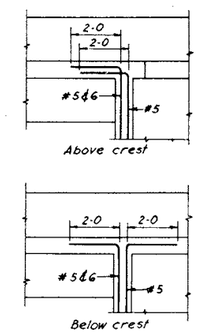
Steel in unexposed face - Vertical steel 2" from face

ELEVATION-WINGWALL



Steel in exposed face - Horizontal steel 2" from face

ELEVATION-WINGWALL



JUNCTION OF SIDEWALL & HEADWALL EXTENSION

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

ENGINEERING STANDARDS UNIT

REFERENCE

ENGINEERING APPROVAL M. M. Culp	CARTOGRAPHIC APPROVAL V. J. Cusey
DESIGNED BY C.A. Reese	DRAWN BY R. Boyeson
CHECKED BY J.R. Brownell	STANDARD DWG. NO. ES-2108-30 BE
DATE 1-13-51	SHEET 3 OF 3 SHEETS

ENGINEERING DRAWING

R/C Drop Spillway F=10-0, h=6-0, L=30-0

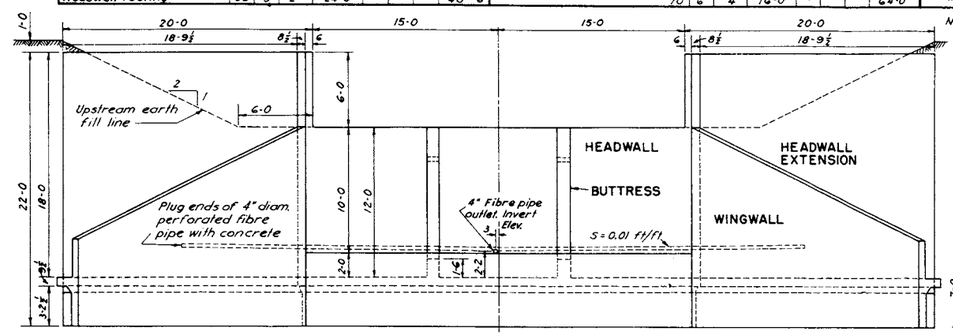
STEEL SCHEDULE

LOCATION	MARK	SIZE	QUAN.	LENGTH	TYPE	A	B	TOTAL FT.
Toewall	1	5	4	8'-0	16	2'-0	6'-0	32'-0
"	2	5	2	23'-3	51			46'-6
"	3	4	2	30'-6	"			61'-0
Toewall and transverse sill	4	3	31	5'-6	"			170'-6
" " " "	5	4	25	5'-6	"			137'-6
Toewall and wingwall	6	5	46	6'-6	"			299'-0
" " "	7	5	18	6'-6	"			117'-0
Wingwall footing	8	4	2	25'-6	"			51'-0
" "	9	4	2	29'-0	"			58'-0
" "	10	4	2	23'-9	"			47'-6
" "	11	4	2	18'-3	"			36'-6
" " " "	12	4	2	12'-3	"			24'-6
Wingwall footing and apron	13	5	8	4'-6	"			36'-0
Toewall and wingwall footing	14	5	30	9'-0	2	3'-6	5'-6	270'-0
" " " "	15	5	8	8'-6	"	3'-6	5'-0	68'-0
" " " "	16	5	8	8'-0	"	3'-6	4'-6	64'-0
" " " "	17	5	4	7'-6	"	3'-6	4'-0	30'-0
" " " "	18	5	4	6'-9	"	3'-6	3'-3	27'-0
" " " "	19	5	4	6'-6	"	3'-6	3'-0	26'-0
" " " "	20	5	6	5'-6	"	3'-6	2'-0	33'-0
Toewall under wingwall	21	4	6	27'-6	16	25'-6	2'-0	165'-0
" " "	22	4	6	28'-0	"	26'-0	2'-0	168'-0
Cutoff wall	23	4	18	24'-0	51			432'-0
Cutoff wall and headwall	24	5	39	6'-0	"			234'-0
" " " "	25	8	4	6'-9	"			27'-0
Cutoff wall and hdwl. ext.	26	5	156	10'-6	"			1638'-0
Cutoff wall and headwall	27	4	23	5'-6	"			126'-6
Apron and sidewall	28	6	68	6'-3	2	3'-3	3'-0	425'-0
Apron	29	5	17	28'-9	51			488'-9
"	30	4	34	3'-6	"			119'-0
Apron and toewall	31	4	24	22'-9	2	21'-9	1'-0	546'-0
Longitudinal sill	32	4	28	6'-3	54	See detail		175'-0
Longitudinal sill and toewall	33	7	4	3'-9	2	2'-9	1'-0	15'-0
Longitudinal sill	34	8	6	8'-0	51			48'-0
Headwall footing	35	5	2	24'-3	"			48'-6

LOCATION	MARK	SIZE	QUAN.	LENGTH	TYPE	A	B	TOTAL FT.
Headwall extension footing	36	4	76	3'-9	51			285'-0
" " "	37	5	4	20'-0	"			80'-0
" " "	38	8	8	10'-3	"			82'-0
" " "	39	5	4	24'-3	"			97'-0
Apron and toewall	40	5	24	22'-3	2	21'-3	1'-0	534'-0
Apron	41	4	32	16'-9	51			536'-0
"	42	4	15	7'-0	"			105'-0
Longitudinal sill	43	8	6	19'-3	"			115'-6
Transverse sill	44	6	4	17'-0	"			68'-0
Buttress	45	5	16	3'-9	"			60'-0
"	46	5	4	2'-9	"			11'-0
Wingwall	47	5	4	10'-9	"			43'-0
"	48	5	4	10'-0	"			40'-0
"	49	5	4	9'-0	"			36'-0
"	50	5	4	8'-3	"			33'-0
"	51	5	4	7'-3	"			29'-0
"	52	5	4	6'-6	"			26'-0
"	53	5	4	5'-6	"			22'-0
"	54	5	4	4'-9	"			19'-0
"	55	5	4	3'-9	"			15'-0
Wingwall and sidewall	56	8	22	9'-9	16	7'-9	2'-0	214'-6
" " "	57	8	2	8'-3	"	6'-3	2'-0	16'-6
" " "	58	8	2	6'-0	"	4'-0	2'-0	12'-0
" " "	59	8	2	3'-9	"	1'-9	2'-0	7'-6
Wingwall	60	5	6	20'-0	51			120'-0
"	61	5	2	19'-0	"			38'-0
"	62	5	2	16'-9	"			33'-6
"	63	5	2	14'-3	"			28'-6
"	64	5	2	12'-0	"			24'-0
"	65	5	2	9'-9	"			19'-6
"	66	5	2	7'-6	"			15'-0
"	67	5	2	5'-0	"			10'-0
Sidewall	68	6	12	17'-6	"			210'-0
"	69	6	2	17'-3	"			34'-6
"	70	6	4	16'-0	"			64'-0

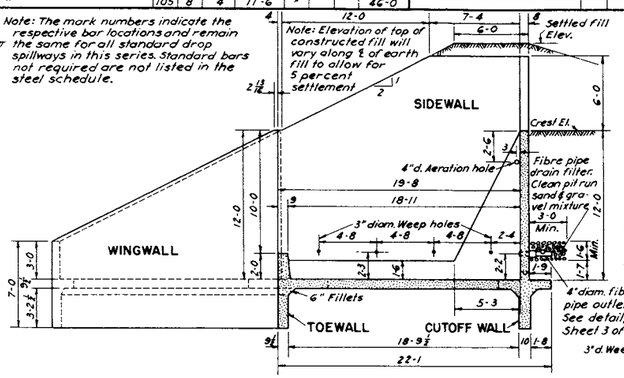
LOCATION	MARK	SIZE	QUAN.	LENGTH	TYPE	A	B	TOTAL FT.
Sidewall	71	6	4	14'-9	51			59'-0
"	72	6	4	13'-9	"			55'-0
"	73	6	4	12'-6	"			50'-0
"	74	6	4	11'-6	"			46'-0
Sidewall and headwall ext.	75	5	26	18'-9	2	16'-9	2'-0	487'-6
" " " "	76	5	2	18'-3	2	16'-3	2'-0	36'-6
" " " "	77	5	2	16'-3	2	14'-3	2'-0	32'-6
" " " "	78	5	2	14'-3	2	12'-3	2'-0	28'-6
" " " "	79	5	2	12'-3	2	10'-3	2'-0	24'-6
" " " "	80	5	2	10'-3	2	8'-3	2'-0	20'-6
" " " "	81	6	34	5'-6	2	3'-6	2'-0	187'-0
Sidewall and wingwall	82	6	46	7'-3	16	5'-3	2'-0	333'-6
Wingwall	83	4	4	10'-6	51			42'-0
"	84	4	4	9'-6	"			38'-0
"	85	4	4	8'-9	"			35'-0
"	86	4	4	8'-0	"			32'-0
"	87	4	4	7'-0	"			28'-0
"	88	4	4	6'-0	"			24'-0
"	89	4	4	5'-3	"			21'-0
"	90	4	4	4'-6	"			18'-0
"	91	4	4	3'-6	"			14'-0
"	92	4	4	2'-9	"			11'-0
"	93	5	4	25'-0	"			100'-0
"	94	5	2	24'-0	"			48'-0
"	95	5	2	20'-6	"			41'-0
"	96	5	2	16'-9	"			33'-6
"	97	5	2	13'-3	"			26'-6
"	98	5	2	9'-9	"			19'-6
"	99	5	2	6'-0	"			12'-0
Wingwall and sidewall	100	5	4	28'-3	16	26'-3	2'-0	113'-0
Headwall extension	101	4	80	12'-6	51			1000'-0
" "	102	3	72	19'-6	"			1404'-0
Headwall	103	3	2	17'-6	"			35'-0
"	104	3	37	11'-6	"			425'-6
"	105	8	4	11'-6	"			46'-0

LOCATION	MARK	SIZE	QUAN.	LENGTH	TYPE	A	B	TOTAL FT.
Headwall	106	4	22	17'-3	51			379'-6
"	107	4	20	2'-9	"			55'-0
"	108	4	20	4'-0	"			80'-0
Sidewall	109	5	14	17'-6	"			245'-0
"	110	5	4	16'-0	"			64'-0
"	111	5	4	15'-0	"			60'-0
"	112	5	4	13'-9	"			55'-0
"	113	5	4	12'-6	"			50'-0
"	114	5	4	11'-6	"			46'-0
Sidewall and headwall	115	5	32	21'-3	2	19'-3	2'-0	680'-0
Sidewall and headwall ext.	116	5	2	20'-0	2	18'-0	2'-0	40'-0
" " " "	117	5	2	18'-6	2	16'-6	2'-0	37'-0
" " " "	118	5	2	17'-3	2	15'-3	2'-0	34'-6
" " " "	119	5	2	15'-6	2	13'-6	2'-0	31'-0
" " " "	120	5	2	14'-3	2	12'-3	2'-0	28'-6
" " " "	121	5	2	12'-9	2	10'-9	2'-0	25'-6
" " " "	122	5	2	11'-3	2	9'-3	2'-0	22'-6
" " " "	123	5	2	9'-9	2	7'-9	2'-0	19'-6
Sidewall and wingwall	124	5	4	15'-6	16	13'-6	2'-0	62'-0
Headwall	125	4	2	17'-6	51			35'-0
"	126	4	21	11'-6	"			241'-6
"	127	4	24	17'-0	"			408'-0
Buttress	128	5	4	8'-6	"			34'-0
"	129	5	4	6'-6	"			26'-0
"	130	5	4	4'-6	"			18'-0
"	131	4	4	4'-9	"			19'-0
"	132	4	4	4'-0	"			16'-0
"	133	4	4	3'-6	"			14'-0
"	134	4	4	2'-9	"			11'-0
"	135	4	4	2'-3	"			9'-0
"	136	4	4	1'-6	"			6'-0
"	137	4	4	11'-3	"			45'-0

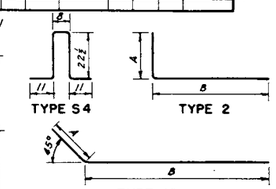


DOWNSTREAM ELEVATION

Note: The mark numbers indicate the respective bar locations and remain the same for all standard drop spillways in this series. Standard bars not required are not listed in the steel schedule.

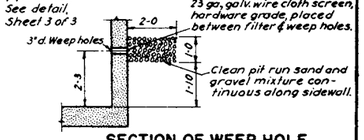


CROSS-SECTION ALONG ℄

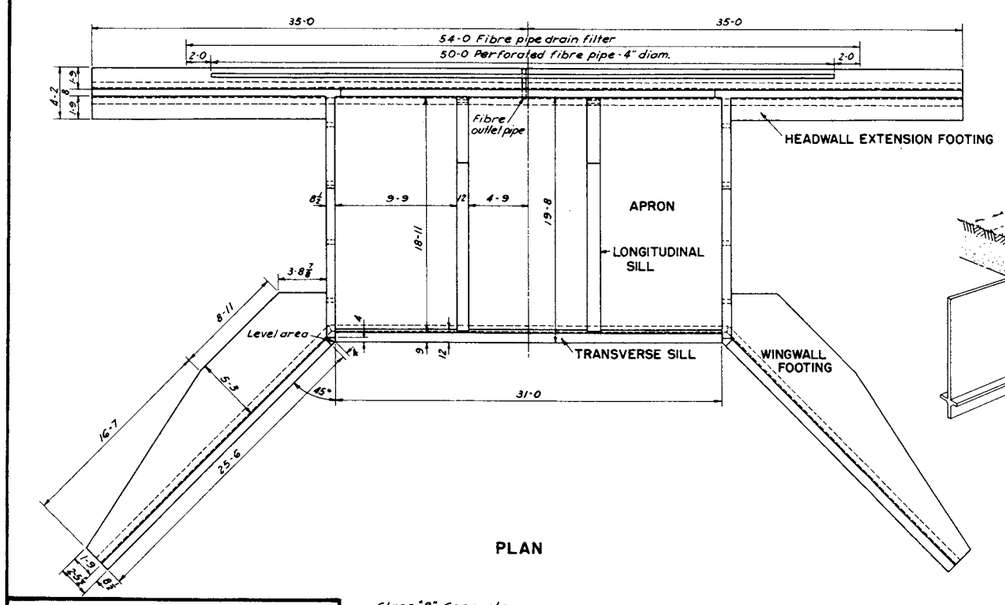


The bar schedule is listed in the approximate order of placement. Bars 1 through 46 are contained in the first pour. All reinforcing steel is round. "St" is an abbreviation for straight.

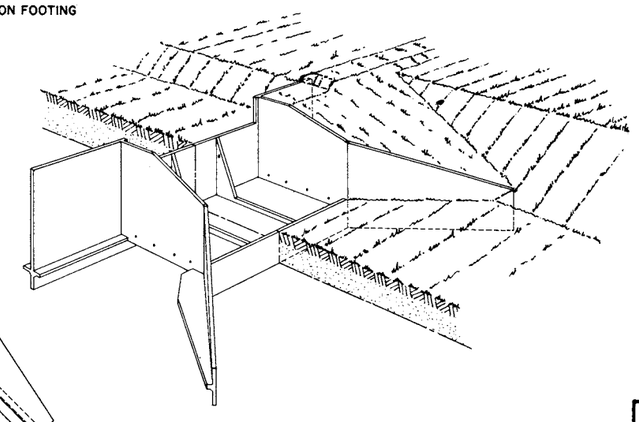
BAR TYPE DETAILS



SECTION OF WEEP HOLE AND DRAIN FILTER



PLAN



PERSPECTIVE VIEW

Not to scale

QUANTITIES

Reinforced concrete		108.45	Cu. yds.
Reinforcing steel			
# 3	2035.00 Ft.	765.16	Pounds
# 4	5655.50 "	3777.87	"
# 5	7361.25 "	7677.78	"
# 6	1532.00 "	2301.06	"
# 7	15.00 "	30.66	"
# 8	569.00 "	1519.23	"
# 9	32.00 "	108.80	"
Drainage - Perforated fibre pipe - 4" d.		52.5	L.F.
Drainage - Drain filter		11.9	Cu. yd.

Class "B" Concrete
f_s = 20,000 lbs/sq. in.
f'_c = 3,000 "
f_c = 1,200 "

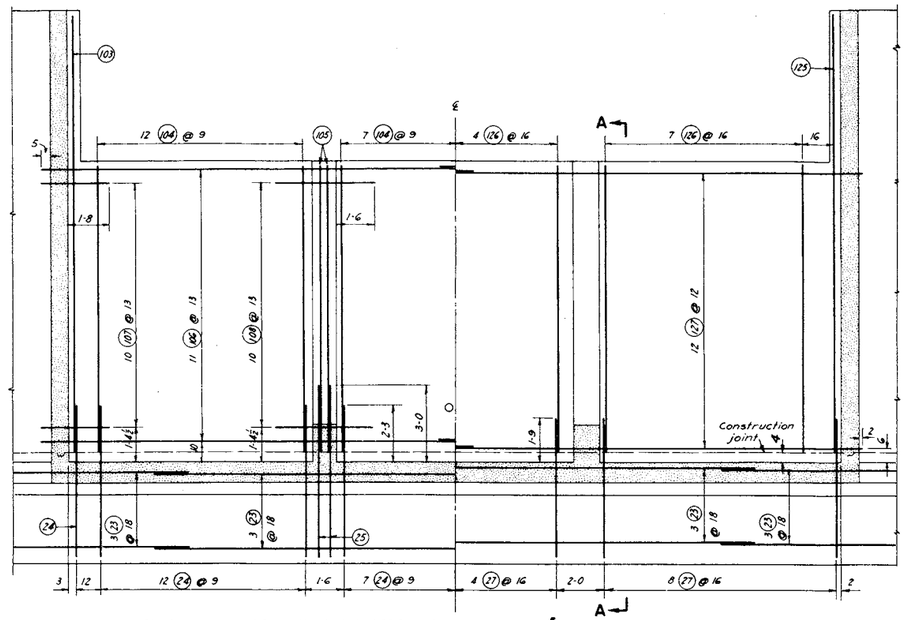
F = 10'-0, h = 6'-0, L = 30'-0

Note: Structure is symmetrical about ℄ except for 4" fibre pipe drain and outlet. Chamfer all exposed edges ¾"

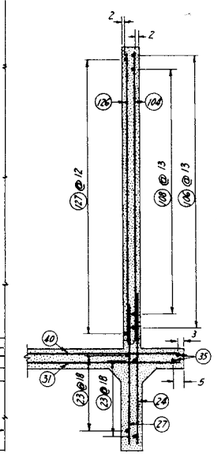
ENGINEERING STANDARDS UNIT	
ENGINEERING APPROVAL: M. M. Culp	CARTOGRAPHIC APPROVAL: Casey
DESIGNED BY: C.A. Reese	DRAWN BY: R. Boyeson
CHECKED BY: J.R. Brownell	STANDARD DWG. NO.: ES-2108-30B
DATE: 2-5-51	SHEET 1 OF 3 SHEETS

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

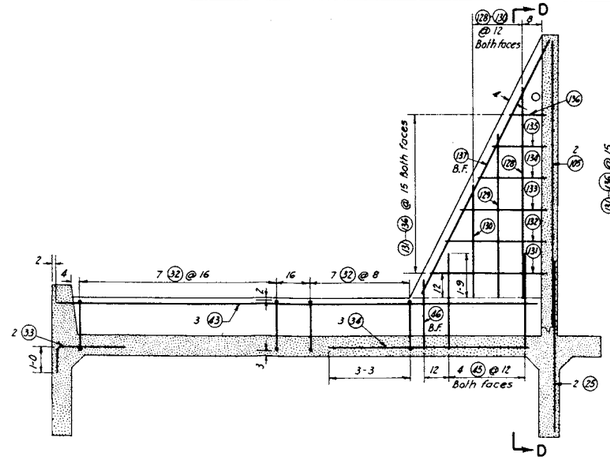
REFERENCE	
ENGINEERING APPROVAL	CARTOGRAPHIC APPROVAL
ADAPTED BY	DRAWN BY
CHECKED BY	DRAWING NO.
DATE	SHEET OF SHEETS



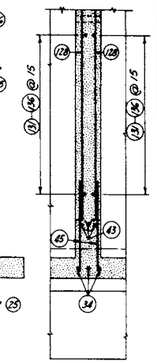
SECTIONAL ELEVATION OF HEADWALL



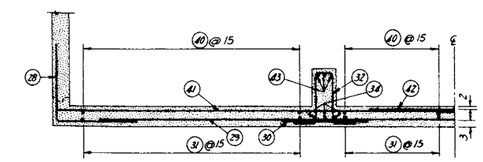
SECTION A-A



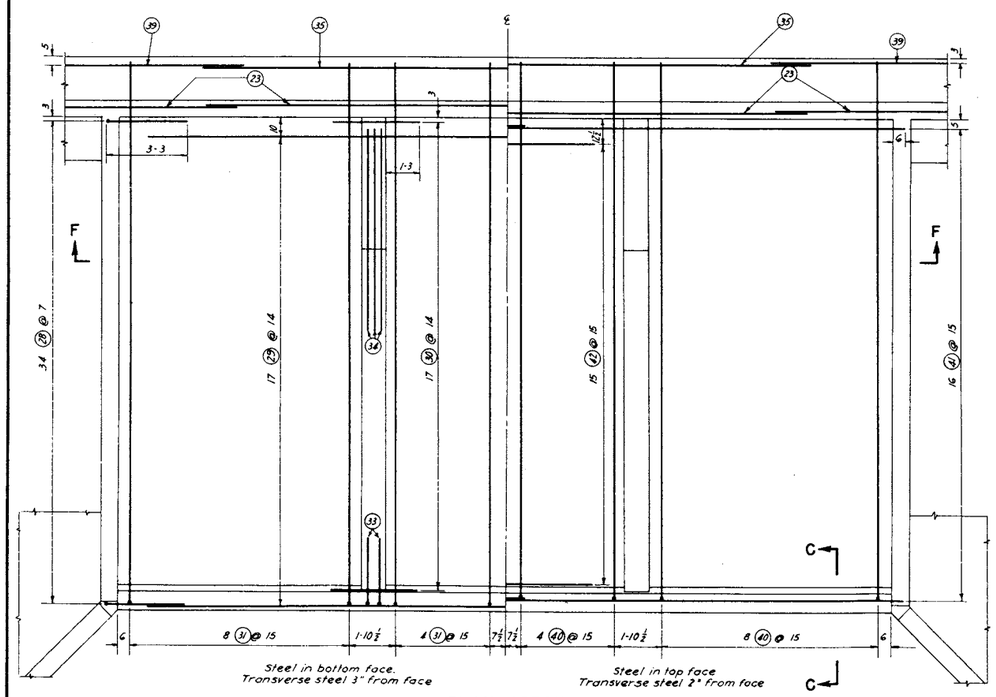
ELEVATION - LONGITUDINAL SILL AND BUTTRESS



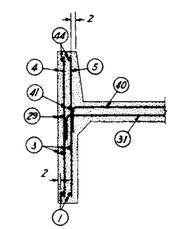
SEC. D-D



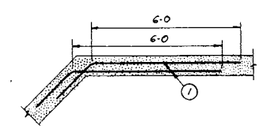
HALF-SECTION F-F



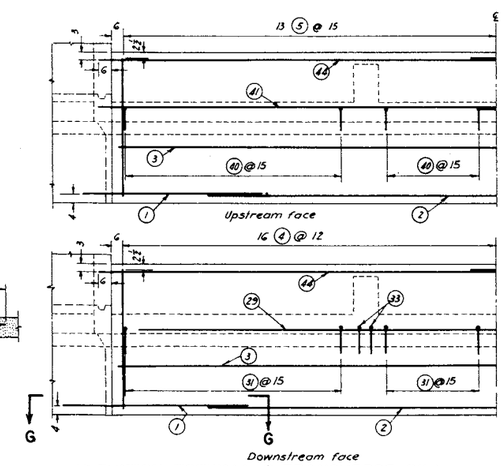
PLAN - APRON



SECTION C-C



SECTION G-G



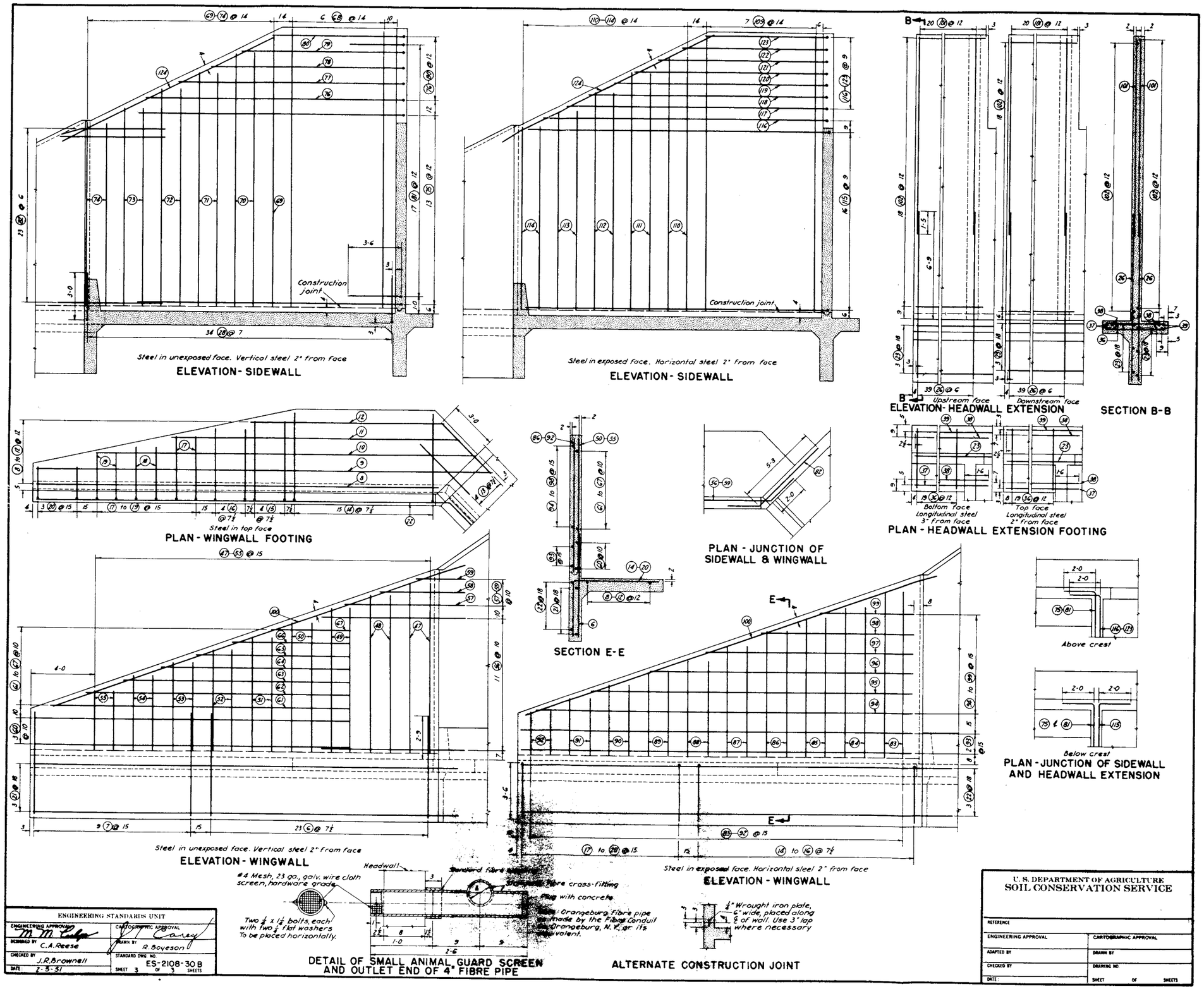
ELEVATION - TOEWALL & TRANSVERSE SILL

Note:
The longitudinal and transverse sills shall be poured monolithically with the apron and without use of a horizontal construction joint.
All dimensions showing distance from face of concrete to steel designate the clear distance.

ENGINEERING STANDARDS UNIT	
ENGINEERING APPROVAL M. M. Culp	CARTOGRAPHIC APPROVAL J. P. Casey
DESIGNED BY C. A. Reese	DRAWN BY R. Boveson
CHECKED BY J. R. Brownell	STANDARD DWG. NO ES-2108-30B
DATE 2-5-51	SHEET 2 OF 3 SHEETS

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

REFERENCE	
ENGINEERING APPROVAL	CARTOGRAPHIC APPROVAL
ADAPTED BY	DRAWN BY
CHECKED BY	DRAWING NO
DATE	SHEET OF SHEETS


Construction joint
Steel in unexposed face. Vertical steel 2" from face
ELEVATION - SIDEWALL
Construction joint
Steel in exposed face. Horizontal steel 2" from face
ELEVATION - SIDEWALL
Upstream face
Downstream face
ELEVATION - HEADWALL EXTENSION
SECTION B-B
Steel in top face
PLAN - WINGWALL FOOTING
PLAN - JUNCTION OF SIDEWALL & WINGWALL
Bottom face Longitudinal steel 3" From face
Top face Longitudinal steel 2" From face
PLAN - HEADWALL EXTENSION FOOTING
SECTION E-E
Above crest
Below crest
PLAN - JUNCTION OF SIDEWALL AND HEADWALL EXTENSION
Steel in unexposed face. Vertical steel 2" from face
ELEVATION - WINGWALL
Steel in exposed face. Horizontal steel 2" from face
ELEVATION - WINGWALL
Headwall
#4 Mesh, 23 ga., galv. wire cloth screen, hardware grade
Two ¼ x 1½ bolts, each with two ¼ flat washers To be placed horizontally.
DETAIL OF SMALL ANIMAL GUARD SCREEN AND OUTLET END OF 4" FIBRE PIPE
¼" Wrought iron plate, 6" wide, placed along ℄ of wall. Use 3" lap where necessary
ALTERNATE CONSTRUCTION JOINT
ENGINEERING STANDARDS UNIT
ENGINEERING APPROVAL
DESIGNED BY C.A.Reese
CHECKED BY J.R.Brownell
DATE 2-5-51
CARTOGRAPHIC APPROVAL
DRAWN BY R.Boyeson
STANDARD DWG. NO. ES-2108-30 B
SHEET 3 OF 3 SHEETS
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
REFERENCE
ENGINEERING APPROVAL
CARTOGRAPHIC APPROVAL
ADAPTED BY
DRAWN BY
CHECKED BY
DRAWING NO.
DATE
SHEET OF SHEETS

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN. AREAS AND PERIMETERS OF BARS AT VARIOUS SPACINGS FOR 12 INCH WIDTH

Areas given in top figures in square inches
Perimeters given in bottom figures in inches

Spacing	Bar Size									Spacing
	#3	#4	#5	#6	#7	#8	#9	#10	#11	
2	0.66 7.07	1.18 9.42	1.84 11.78	2.65 14.14						2
2 1/4	0.59 6.28	1.05 8.38	1.64 10.47	2.36 12.57	3.21 14.66					2 1/4
2 1/2	0.53 5.65	0.94 7.54	1.47 9.42	2.12 11.31	2.89 13.20	3.77 15.08				2 1/2
2 3/4	0.48 5.14	0.86 6.85	1.34 8.57	1.93 10.28	2.62 12.00	3.43 13.71				2 3/4
3	0.44 4.71	0.79 6.28	1.23 7.85	1.77 9.42	2.40 11.00	3.14 12.57	4.00 14.18			3
3 1/4	0.41 4.35	0.73 5.80	1.13 7.25	1.63 8.70	2.22 10.15	2.90 11.60	3.69 13.09			3 1/4
3 1/2	0.38 4.04	0.67 5.39	1.05 6.73	1.51 8.08	2.06 9.42	2.69 10.77	3.43 12.15	4.34 13.68		3 1/2
3 3/4	0.35 3.77	0.63 5.03	0.98 6.28	1.41 7.54	1.92 8.80	2.51 10.05	3.20 11.34	4.05 12.77	5.00 14.18	3 3/4
4	0.33 3.53	0.59 4.71	0.92 5.89	1.33 7.07	1.80 8.25	2.36 9.42	3.00 10.63	3.80 11.97	4.69 13.29	4
4 1/4	0.31 3.33	0.55 4.44	0.87 5.54	1.25 6.65	1.70 7.76	2.22 8.87	2.82 10.01	3.57 11.27	4.41 12.51	4 1/4
4 1/2	0.29 3.14	0.52 4.19	0.82 5.24	1.18 6.28	1.60 7.33	2.09 8.38	2.67 9.45	3.37 10.64	4.17 11.81	4 1/2
4 3/4	0.28 2.98	0.50 3.97	0.78 4.96	1.12 5.96	1.50 6.94	1.98 7.94	2.53 8.95	3.20 10.08	3.95 11.19	4 3/4
5	0.26 2.83	0.47 3.77	0.74 4.71	1.06 5.65	1.44 6.60	1.88 7.54	2.40 8.51	3.04 9.58	3.75 10.63	5
5 1/4	0.25 2.69	0.45 3.59	0.70 4.49	1.01 5.39	1.37 6.28	1.80 7.18	2.29 8.10	2.89 9.12	3.57 10.13	5 1/4
5 1/2	0.24 2.57	0.43 3.43	0.67 4.28	0.96 5.14	1.31 6.00	1.71 6.85	2.18 7.73	2.76 8.71	3.41 9.66	5 1/2
5 3/4	0.23 2.46	0.41 3.28	0.64 4.10	0.92 4.92	1.25 5.74	1.64 6.56	2.09 7.40	2.64 8.33	3.26 9.25	5 3/4
6	0.22 2.36	0.39 3.14	0.61 3.93	0.88 4.71	1.20 5.50	1.57 6.28	2.00 7.09	2.53 7.98	3.12 8.86	6
6 1/2	0.20 2.18	0.36 2.90	0.57 3.62	0.82 4.35	1.11 5.07	1.45 5.80	1.85 6.54	2.34 7.37	2.88 8.18	6 1/2
7	0.19 2.02	0.34 2.69	0.53 3.37	0.76 4.04	1.03 4.71	1.35 5.39	1.71 6.08	2.17 6.84	2.68 7.59	7
7 1/2	0.18 1.89	0.31 2.51	0.49 3.14	0.71 3.77	0.96 4.40	1.26 5.03	1.60 5.67	2.03 6.38	2.50 7.09	7 1/2
8	0.17 1.77	0.29 2.36	0.46 2.95	0.66 3.53	0.90 4.12	1.18 4.71	1.50 5.32	1.90 5.99	2.34 6.65	8
8 1/2	0.16 1.66	0.28 2.22	0.43 2.77	0.62 3.33	0.85 3.88	1.11 4.44	1.41 5.00	1.79 5.63	2.21 6.25	8 1/2
9	0.15 1.57	0.26 2.09	0.41 2.62	0.59 3.14	0.80 3.67	1.05 4.19	1.33 4.73	1.69 5.32	2.08 5.91	9
9 1/2	0.14 1.49	0.25 1.98	0.39 2.48	0.56 2.98	0.76 3.47	0.99 3.97	1.26 4.48	1.60 5.04	1.97 5.60	9 1/2
10	0.13 1.41	0.24 1.88	0.37 2.36	0.53 2.83	0.72 3.30	0.94 3.77	1.20 4.25	1.52 4.79	1.88 5.32	10
10 1/2	0.13 1.35	0.22 1.80	0.35 2.24	0.50 2.69	0.69 3.14	0.90 3.59	1.14 4.05	1.45 4.56	1.79 5.06	10 1/2
11	0.12 1.29	0.21 1.71	0.33 2.14	0.48 2.57	0.66 3.00	0.86 3.43	1.09 3.87	1.38 4.35	1.70 4.83	11
11 1/2	0.12 1.23	0.20 1.64	0.32 2.05	0.46 2.46	0.63 2.87	0.82 3.28	1.04 3.70	1.32 4.16	1.63 4.62	11 1/2
12	0.11 1.18	0.20 1.57	0.31 1.96	0.44 2.36	0.60 2.75	0.79 3.14	1.00 3.54	1.27 3.99	1.56 4.43	12
13	0.10 1.09	0.18 1.45	0.28 1.81	0.41 2.18	0.56 2.54	0.72 2.90	0.92 3.27	1.17 3.68	1.44 4.09	13
14	0.09 1.01	0.17 1.35	0.26 1.68	0.38 2.02	0.52 2.36	0.67 2.69	0.86 3.04	1.08 3.42	1.34 3.80	14
15	0.09 0.94	0.16 1.26	0.25 1.57	0.35 1.89	0.48 2.20	0.63 2.51	0.80 2.84	1.01 3.19	1.25 3.54	15
16	0.08 0.88	0.15 1.18	0.23 1.47	0.33 1.77	0.45 2.06	0.59 2.36	0.75 2.66	0.95 2.99	1.17 3.32	16
17	0.08 0.83	0.14 1.11	0.22 1.39	0.31 1.66	0.42 1.94	0.55 2.22	0.71 2.50	0.89 2.82	1.10 3.13	17
18	0.07 0.79	0.13 1.05	0.20 1.31	0.29 1.57	0.40 1.83	0.52 2.09	0.67 2.36	0.84 2.66	1.04 2.95	18
	#3	#4	#5	#6	#7	#8	#9	#10	#11	

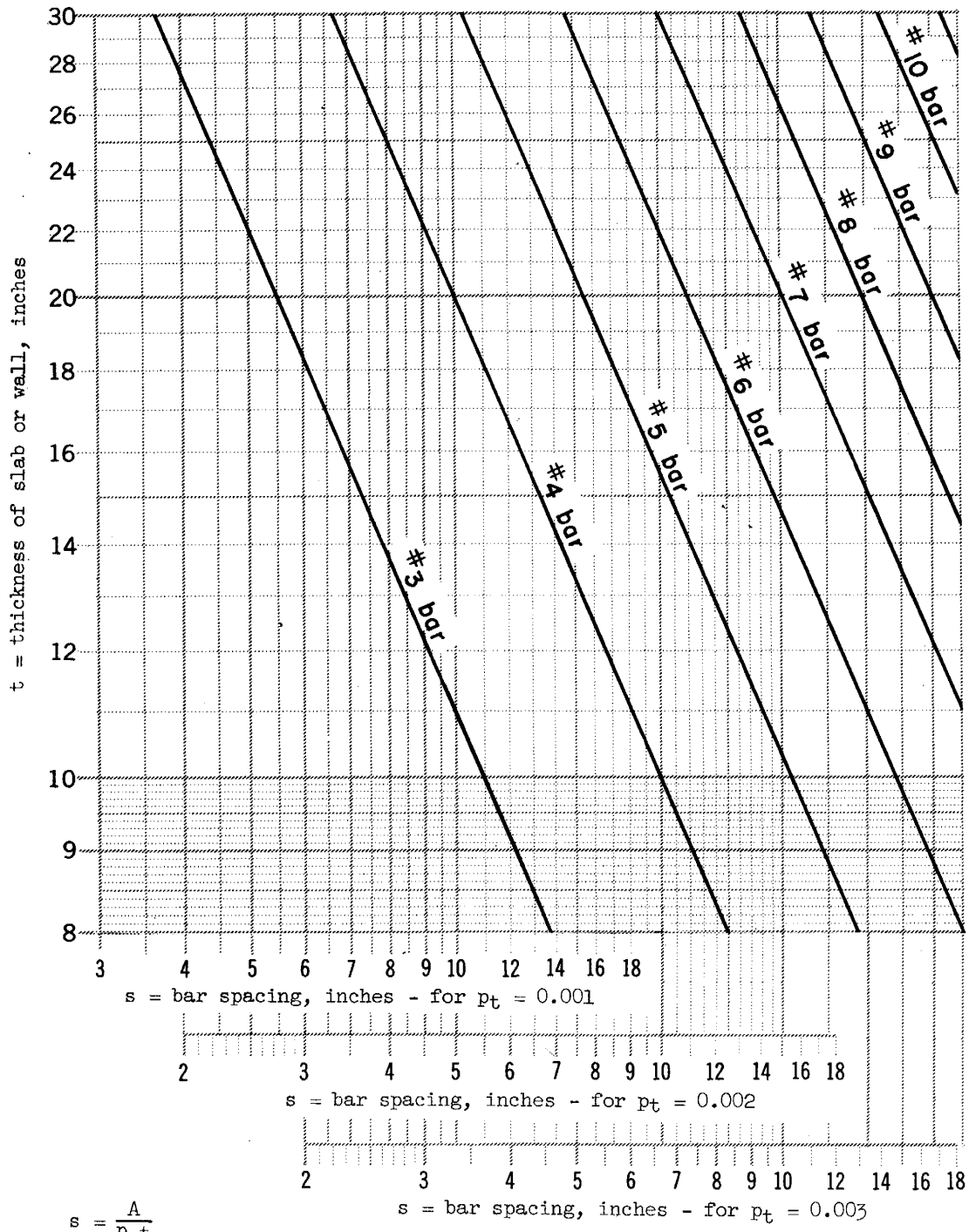
REFERENCE

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES – 46
SHEET 1 OF 1
DATE 12-14-50

REVISED 7-64

STRUCTURAL DESIGN: REINFORCED CONCRETE DESIGN
BAR SPACINGS FOR TEMPERATURE AND SHRINKAGE STEEL



$$s = \frac{A}{p_t t}$$

A = area of bar, sq in.

p_t = temperature and shrinkage steel ratio

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - DESIGN SECTION	STANDARD DWG. NO. ES- 47 SHEET 1 OF 1 DATE 12-8-50

REVISED 7-64

4.4 Detailing

4.4.1 General. Structural drawings should be simple, clear, complete, and accurate. They should not contain unnecessary lines, dimensions, symbols, or notes. They should contain, however, all the essential information supplemented by an adequate set of notes that are clearly stated. Such drawings save time and expense in the drafting room and on the job. Any engineering office that is doing an appreciable amount of reinforced concrete design and/or detailing should have a copy of the "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI Standard 315-48)" published by the American Concrete Institute.

4.4.2 Drawing Standards. The standard drawing sizes and title blocks to be used by the Engineering Standards Unit are shown in drawing ES-16.

The recommended scales for detailing are: 1/4" = 1'-0", 3/8" = 1'-0", 1/2" = 1'-0", 3/4" = 1'-0", and 1" = 1'-0". The scale selected will depend on the proper layout of the drawing and the scale necessary to clearly show all details.

The use of symbols and abbreviations reduces the required drafting time and makes the drawing more clear and simple. The symbols and abbreviations should be clear in meaning and should above all be consistent. See drawings ES-2108-30BE and ES-2108-30B for the use of recommended symbols and abbreviations.

The recommended type and weight of line to accent the important features are as follows:

Concrete line	1/100"
Unexposed concrete line	1/100"
Reinforcement	1/50"
Center lines	1/100"
Dimension lines	1/150"

4.4.3 Engineering Drawing. The engineering drawing is prepared by the designer to convey all the necessary information to the draftsman so that he can prepare the detailed structure drawing. The engineering drawing should be simple, but should be clear and complete enough so that the draftsman will have no doubts as to the designer's intentions. See drawing ES-2108-30BE for a sample engineering drawing.

4.4.4 Detailed Structure Drawing. The detailed structure drawing is prepared by the draftsman and will be used by the steel fabricator, construction engineer, and the contractor. This drawing must be complete and accurate. It should contain the necessary information only. Superfluous material and notes add nothing to a drawing but confusion and higher

costs. The first sheet of this drawing will be the layout sheet which will show the over-all structure features and dimensions. It is usually advantageous to show a list of materials on the layout sheet. The following detail sheets will show the steel placement, but will not show the concrete dimensions except in special cases. The bar schedule may be shown on the layout sheet, all on one detail sheet, or may be divided on two or more of the detail sheets, whichever provides the better layout. The bars should be listed in the order that they will be placed, or as nearly so as possible. See drawing ES-2108-30B for a sample detailed structure drawing.

4.4.5 Reinforcement. Typical bar types that will be used are shown on drawing ES-18.

The required tension lap splices and required bar extensions for various size bars for the classes of concrete are shown in drawings ES-227 and ES-160 respectively.

The details of hook bars are shown in drawing ES-20. The recommended sizes for 180° hooks should be used wherever possible.

4.4.6 Notes to Designers and Detailers. The following notes are quoted, with permission, from the "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI Standard 315-48)."

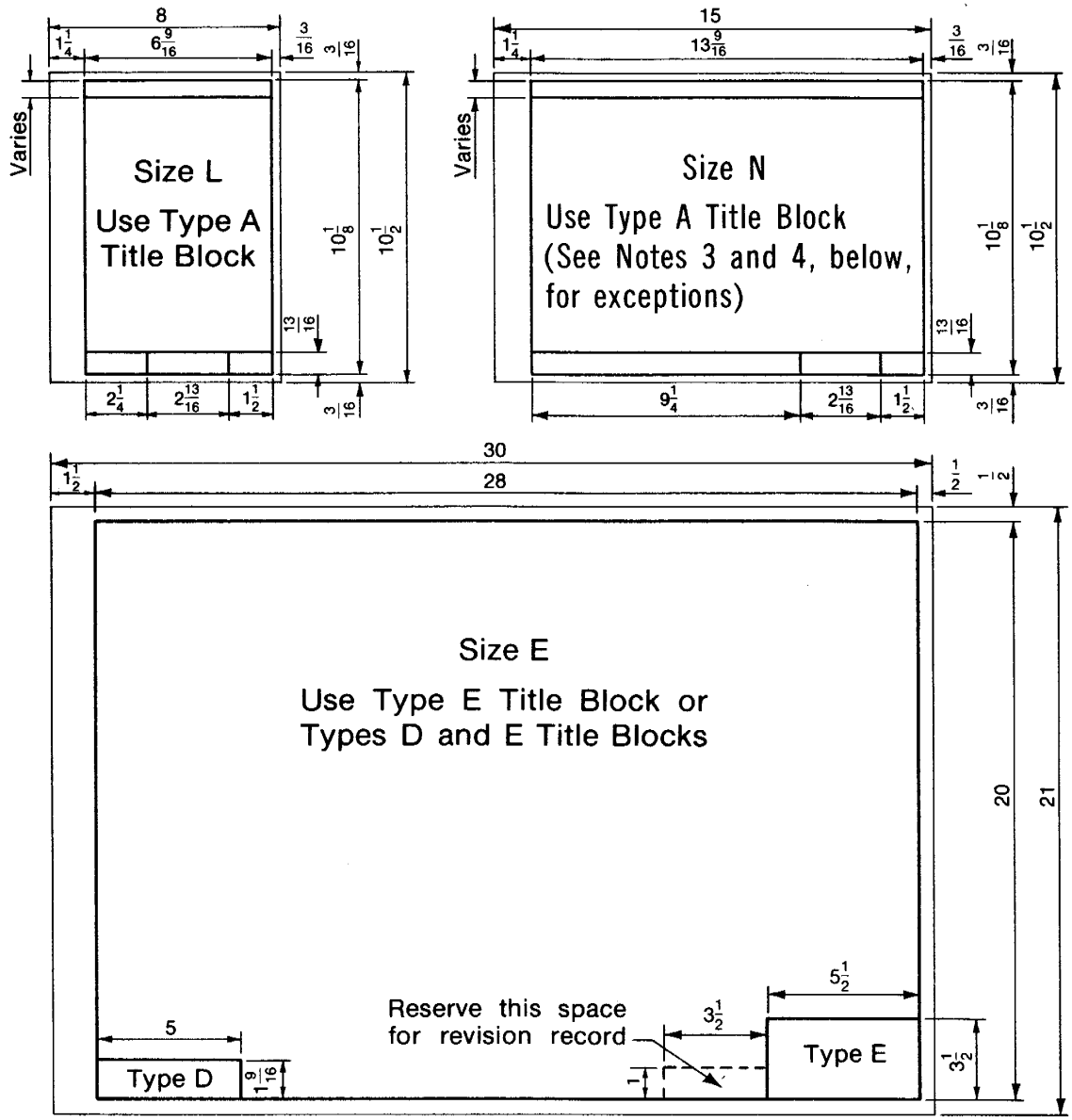
"109. Notes to Designers and Detailers. The designer must show plans which are complete and clear in every detail. The detailer should not be expected to do any designing or to guess at what the designer desires. The discussion in sections 105 and 106, together with the following notes, will assist in preparing satisfactory drawings.

"(a) Plan Study. The detailer should completely understand the engineer's design and the builder's method of construction before be begins detailing the structure. The preparation of placing drawings, bending details and bar lists for reinforcing steel requires careful study of the engineer's plans, thoughtful planning, patience, and above all, accuracy. This statement cannot be overemphasized.

"(b) Planning. The next step for the detailer in planning his work should be to determine the scale of the drawings he is to make, the size of his drawings, their arrangements with regard to plans, sections, details and schedules, and the number of sheets required. In his planning he should bear in mind the successive steps of construction so that all steel required for building any portion of the structure will be shown on the plan for that portion. If possible, the drawings should be so planned that the reinforcing steel setter will need but one print at a time. Where a structure is large and more than one drawing must be employed, the structure should be divided into sections in order to keep the drawings on a practical scale.

"(c) Checking. Placing drawings should be checked by the designing engineer before the reinforcing steel is fabricated.

STRUCTURAL DESIGN: STANDARD DRAWING SIZES AND TITLE BLOCKS



NOTES:

1. All drawings shall be one of the above sizes and shall have border and trim lines.
2. Standard size typewritten material placed on Size L and Size N drawings shall not have a linear reduction greater than 1 to 0.75 in the final form.
3. Size L and Size N drawings prepared for inclusion in National Engineering Technical Material shall use the Type A Title Block, except where a Size E drawing is reduced to a Size N. A Type A Title Block consists of two parts, one at the top of the sheet and the other at the bottom.
4. Size N drawings prepared for a purpose other than inclusion in National Engineering Technical Material may use the Type F Title Block.
5. All Size E drawings shall be prepared to accept a linear reduction of 1 to 0.5.
6. Type E Title Blocks shall be used on all Size E drawings.
7. Type D and E Title Blocks shall be used on all National Standard Detail Drawings that are to be incorporated into a set of construction plans. Type D shall be placed in the lower left-hand corner of the drawing and Type E in the lower right-hand corner. The Type D Title Block shall be completed by the office preparing the original standard drawing and the Type E by the office using the standard.
8. As shown above, all Size E drawings shall be prepared with a 3½ x 1 inch vacant space (without border lines) for recording drawing revisions..

REFERENCE	U.S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - DESIGN UNIT	STANDARD DWG. NO. ES- 16 SHEET 1 OF 2 DATE 4-7-50

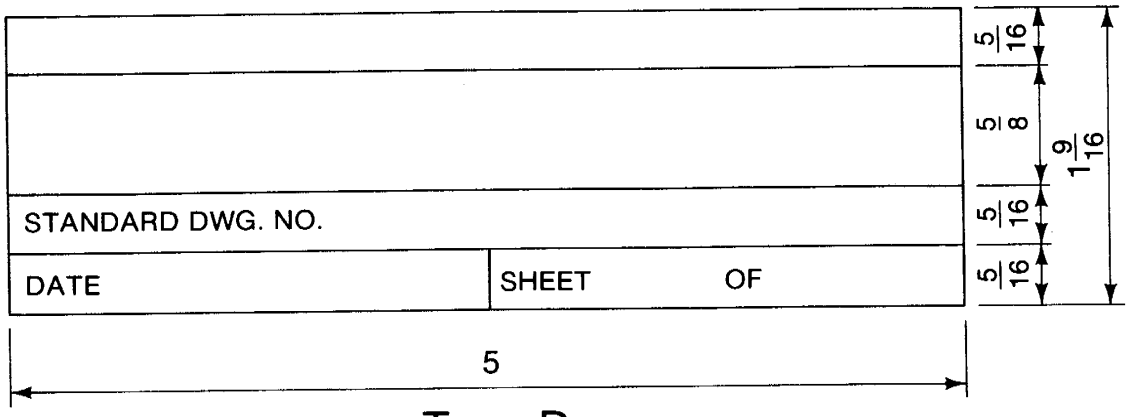
REVISED 9-76

STRUCTURAL DESIGN: STANDARD DRAWING SIZES AND TITLE BLOCKS

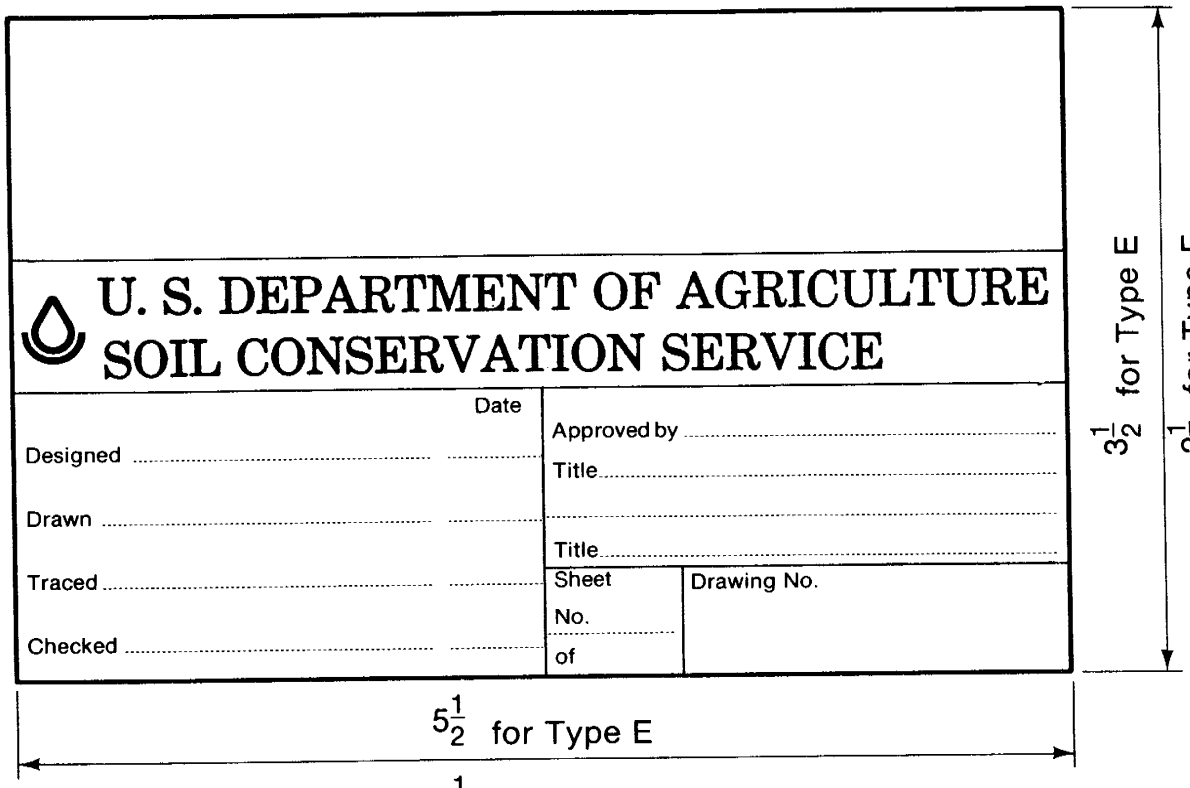
Type A

The title block used at the top and bottom of this sheet is Type A

For a Type A Title Block, enter name of office preparing the drawing in center block at bottom of page.



Type D



Type E and Type F

REFERENCE	U.S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - DESIGN UNIT	STANDARD DWG. NO. ES- 16 SHEET 2 OF 2 DATE 4-7-50

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