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ENGINEERING EXPERIMENT STATION
OREGON STATE COLLEGE
CORVALLIS, OREGON

REVIEW OF ENGINEERING REGISTRATION
2. CIVIL ENGINEERING

By

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Oregon State College

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FOREWORD

This publication is the second in a series of circulars designed to assist the graduate engineer in reviewing engineering subject matter in order to prepare for registration examinations. As in the first publication of the series, most of the illustrative problems are taken from recent examinations of the Oregon State Board of Engineering Examiners.

It is beyond the scope of these publications to give a detailed explanation of theoretical considerations and methods of analysis. For those who require additional study of specific subject material, a list of textbook references is included at the end.

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REVIEW FOR ENGINEERING REGISTRATION
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I. STRUCTURAL ANALYSIS

Evaluation of internal loads and stresses through use of principles of statics and strength of materials is a necessary preliminary to the actual design of any structure. It is not practical, and sometimes not possible, to analyze structures as constructed because of the effects of continuity and friction. The analysis is made on an idealized structure which must represent as nearly as possible the field conditions.

For example, consider the following assumptions that are necessary in the analysis of a truss:

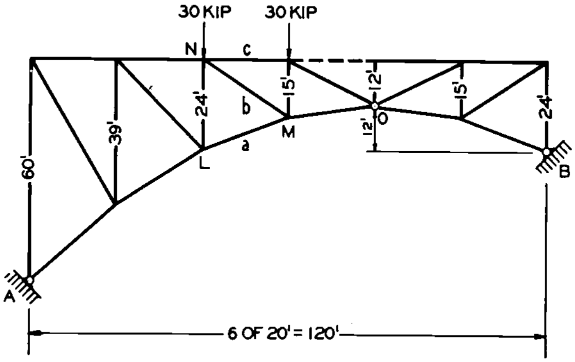
1. All joints in a truss are connected by frictionless pins (pin-connected).
2. Loads are applied only at the joints.

The assumptions are not precisely correct for riveted or welded trusses, but through the use of these assumptions an analysis of the truss can be made using only the laws of statics. The results obtained are somewhat in error as compared to the action of the actual structure, but generally are close enough to the truth for design purposes.

Example: Forces in Truss Members

Calculate the forces in members a, b, and c of the three-hinged trussed arch with two 30-kip loads, as shown.

A change in temperature causes a horizontal force of 40 kips at A. What will be the force in a, b, and c due to this horizontal load?



Balance the external forces acting on the arch by noting that the right-hand portion of the arch is not loaded. Therefore, the reaction of B must pass through point 0, and

$$B_V = \frac{12}{40} B_H$$

$$0 = 30(40) + 30(60) - 36\,B_H - 120\,B_V \qquad (\Sigma M_A = 0)$$

$$72\,B_H = 3000$$

$$A_H = B_H = 41.7 \text{ kip}$$

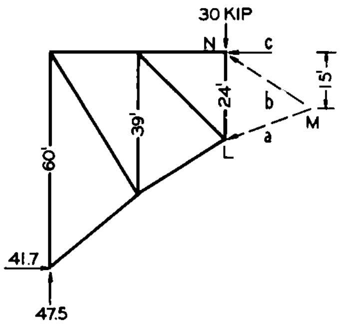
$$B_V = 12.5 \text{ kip}, \; A_V = \underline{\underline{47.5 \text{ kip}}}$$

Check: $\quad 0 = 30(60)+30(80)+41.7(36)-47.5(120) \quad (\Sigma M_B = 0)$

To find the internal forces acting in members a, b, and c, cut a section between L and M and replace the cut members with the internal forces acting along the axes of the members.

$$\Sigma M_M = 0 = 30(20) + 41.7(45) - 47.5(60) + 15c$$

$$c = \frac{370}{15} = \underline{\underline{24.7 \text{ kip}}} \text{ (compression)}$$



$$\Sigma M_N = 0 = 41.7(60) - 47.5(40) - 24a_H$$

$$a_H = \frac{600}{24} = 25 \text{ kip}$$

$$A_V = 11.3 \text{ kip}$$

$$a = 25(\frac{22}{20}) = \underline{\underline{27.5 \text{ kip}}} \text{ (compr)}$$

$$\Sigma M_L = 0 = 24.7(24) + 41.7(36)$$
$$- 47.5(40) - 24\, b_H = 0$$

$$b_H = \frac{194}{24} = 8.1 \qquad b_V = 6.1$$

$$b = 8.1(\frac{5}{4}) = \underline{\underline{10.1 \text{ kip}}} \text{ (tension)}$$

Check:

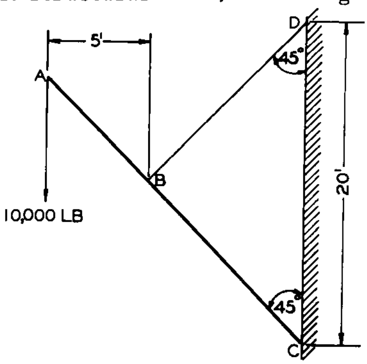
$\Sigma F_V = 0 = 47.5 - 30 - 11.3 - 6.1$

$\Sigma F_H = 0 = 41.7 - 24.7 + 8.1 - 25$

Some structures, although they are pin-connected, are not classed as trusses because the loads are not applied at the joints. In this case, some of the members may be subjected to bending as well as to axial loads. In an idealized truss, members are subjected only to axial loads.

Example: Design of Frame Member

The structural frame shown is pin-connected and supports a vertical live load of 10,000 pounds. Design the member A-B-C of structural steel, assuming the given load to include impact.



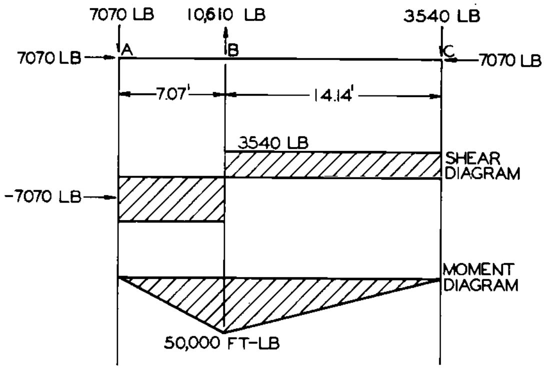
$$\Sigma M_D = 0 = 10(15) - C_H(20)$$

$$C_H = 7.5 \text{ kip}$$

$$D_H = D_V = 7.5 \text{ kip}$$

$$C_V = 2.5 \text{ kip}$$

$$DB = 7.5\ (1.414) = 10.6 \text{ kip}$$



Member ABC must be designed for

Shear = 7070 lb
Moment = 50, 000 ft-lb
Axial load = 7070 lb

If the structure to be designed is a three-dimensional figure rather than two-dimensional, as has been considered thus far, the three equations of static equilibrium are increased to six:

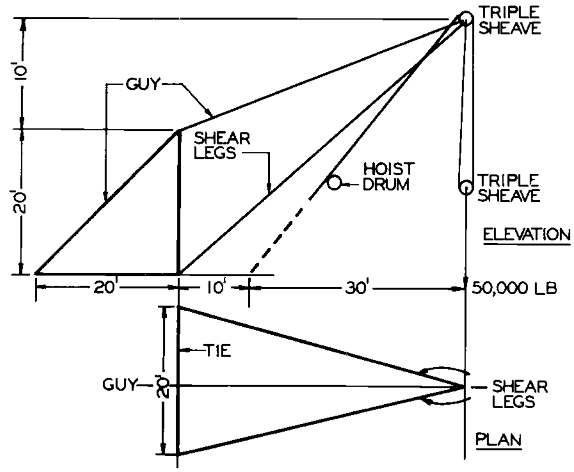
$$\Sigma F_x = 0$$
$$\Sigma F_y = 0$$
$$\Sigma F_z = 0$$
$$\Sigma M_{x-y} = 0$$
$$\Sigma M_{x-z} = 0$$
$$\Sigma M_{y-z} = 0$$

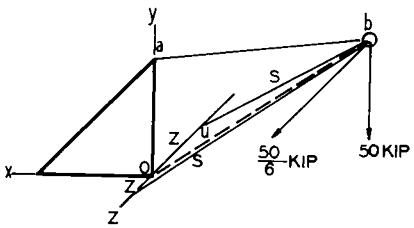
The x, y, and z axes are any three axes that are mutually perpendicular. The forces along any of these axes must sum to zero, and the sum of moments in the three planes defined by these axes also must be zero.

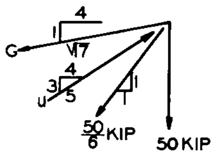
Example: Structural Design

The shear legs sketched below are to be located on a dock for handling cargo.

1. What are the stresses in the various members and in the line?
2. Make a preliminary design for the legs.
3. Sketch the principal connection details.







View in x-y plane at b:

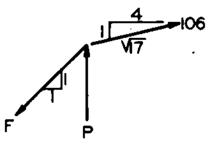
$$\Sigma F_x = 0 = \frac{4}{\sqrt{17}}G + \frac{50}{6}(0.707) \qquad - \frac{4}{5}u \quad (1)$$

$$\Sigma F_y = 0 = \frac{1}{\sqrt{17}}G + \frac{50}{6}(0.707)+50 - \frac{3}{5}u \quad (2)$$

$$\Sigma F_x = 0 = \frac{1}{\sqrt{17}}G + \frac{50}{24}(0.707) \qquad - \frac{1}{5}u \quad (1)$$

$$0 = \frac{3}{24}(50)(0.707) + 50 \qquad - \frac{2}{5}u$$

$\underline{\underline{u = 136 \text{ kip}}}$ $\qquad$ $\underline{\underline{G = 106 \text{ kip}}}$

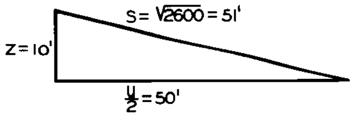


View in x-y plane at a:

$F_x = F_y = 106\ (4/\sqrt{17}) = 103$ kip

$F = \underline{\underline{145 \text{ kip}}}$

$P = 103 - 106\ (1/\sqrt{17}) = \underline{\underline{77.3 \text{ kip}}}$



View in Z-u plane for stress in shear leg:

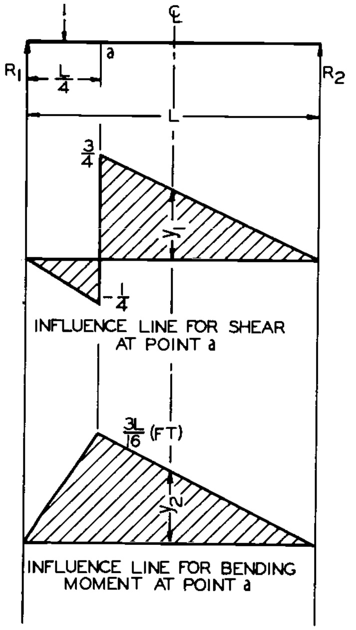
$S = \frac{136}{2}(\frac{51}{50}) = \underline{\underline{69.5 \text{ kip}}}$

$Z = \frac{136}{2}(\frac{10}{50}) = \underline{\underline{13.6 \text{ kip}}}$

Shear and bending moment diagrams show the shear or moment at <u>all points</u> in a member for a fixed loading. It is often desirable to investigate the effects of a moving load on the shear and moment at some given point in the member. An <u>influence line</u> shows the variation in shear or moment at <u>one point</u> in the member for <u>all</u> positions of a unit load as it moves across the span.

Consider a simple beam and construct influence lines for shear and bending moment at the quarter point as a unit load moves from left to right across the span.

If the unit load is over R_1, the shear at a is zero, and the initial ordinate to the influence line at R_1 is zero. As the unit load moves out on the span, and until it reaches point a, the shear at point a is numerically equal to the reaction R_2, and is negative in sign. (Sketch a shear diagram for the beam with the unit load at L/8, and check this statement.)



The shear at a increases as R_2 increases and reaches a maximum negative value when the unit load is just to the left of point a, at which time $R_2 = 1/4$. As the unit load moves over point a, the shear changes sign and becomes equal to R_1 for any position of the unit load between a and R_2. When the unit load is just to the right of point a, the shear at point a is equal to $R_1 = 3/4$. When the unit load is at the center of the beam, the shear at point a still varies with R_1, and $R_1 = 1/2$ in this case.

Note that the ordinates to the influence diagram have no dimensions because the unit load has none.

In constructing the influence line for bending moment, note that when the unit load is between R_1 and a, the moment at a is $(3/4\ L)(R_2)$, and the moment varies with R_2 from zero when the unit load is at R_1 to a maximum of 3L/16 when the unit load is at point a. When the unit load is between a and R_2, the bending moment at a is $(1/4\ L)(R_1)$ and varies with R_1 as the unit load moves. The ordinate to the influence diagram at midspan would thus be $(1/4\ L)(1/2) = 1/8\ L$.

Assume we wish to know the shear at point a if a 10-kip load is placed at midspan. From the influence line just constructed, the shear at point a due to a unit load at midspan is $y_1 = 1/2$. Therefore, the shear at a due to the 10-kip load at midspan would be 10 kip(y_2 ft) = 10 kip(1/8 L ft) = 1.25 L ft-kip. Thus the shear or moment at point a due to a concentrated load at any point in the span is given by multiplying the load by the ordinate to the influence line under the load.

If the beam is loaded with some uniformly distributed load w lb/ft, this loading can be expressed as a series of small concentrated loads of the magnitude w dx, and each of these loads multiplied by its corresponding ordinate would give the increment of shear at point a due to a uniform load over the small length of span dx. The total shear at a would be the sum of the increments

$$dV = w \; dx \; y$$

$$V = \int w \; y \; dx$$

$$W = w \int y \; dx$$

The term following the integral sign above is the area under the influence line. Therefore, the shear or bending moment at point a due to a uniformly distributed load over all or part of the span may be found by multiplying the intensity of loading w by the area under the influence line for that portion of the span which is loaded. Note that for the beam shown a maximum shear at a results when only the portion of the span between a and R_2 is loaded.

Example: Steel Bridge

The following is given for a steel bridge:

Uniform total dead load	= 1000 lb/ft (udl)
Uniform moving live load	= 550 lb/ft (ull)
Moving concentrated live load	= 15,000 lb (cll)

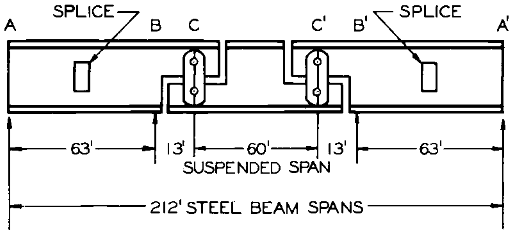
Find the following:

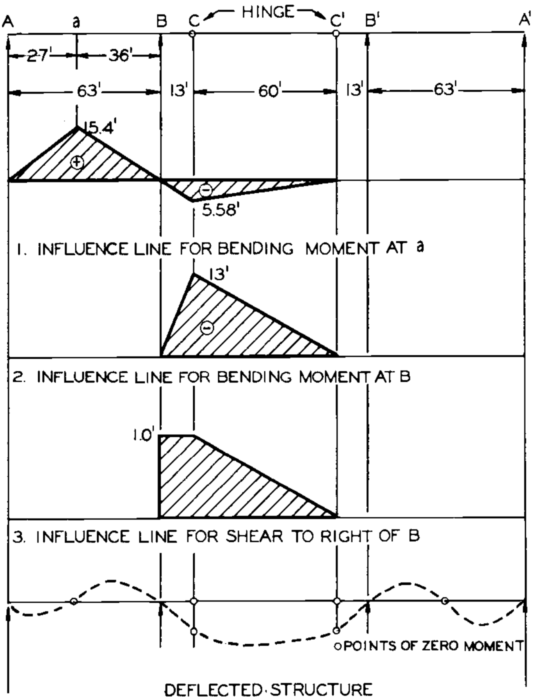
1. Maximum moments in ft-lb at
   a) 27 ft from A in AB
   b) B
   c) C-C'

2. Maximum shear at B in BC.

3. Select a wide flange beam section for the moment computed in 1c for a maximum fiber stress of 18,000 psi.

4. Sketch the deflected shape of the structure by means of a line diagram, with ull from B to B' and the cll at C.





(1) Udl on entire structure:

$$M_a = 1.0\left[15.4(\frac{63}{2}) - 5.58(\frac{73}{2})\right] = 281$$

Ull on span A-B only:

$$M_a = 0.55\left[15.4(\frac{63}{2})\right] = 267$$

Cll at a:

$$M_a = 15.4(15) = \underline{231}$$

Max moment at a $= 779$ kip-ft

$= \underline{\underline{779,000 \text{ ft-lb}}}$

(2) Udl and ull over B-C':

$$1.55\left[13(\frac{73}{2})\right] = 735 \text{ kip-ft}$$

Cll at C:

$$15(13) = \underline{195} \text{ kip-ft}$$

Max moment at B $= 930$ kip-ft

$= \underline{\underline{930,000 \text{ ft-lb}}}$

(3) Udl and ull over B-C':

$$1.55(13 + \frac{60}{2}) = 66.7 \text{ kip}$$

Cll at B or C:

$$15(1) = \underline{15.0} \text{ kip}$$

Max shear at B $= \underline{\underline{81.7 \text{ kip}}}$

Max moment in span C-C' at center when:

Udl and ull over C-C'

$$M_{℄} = \frac{1.55(60)^2(1000)}{8} = 697,000 \text{ ft-lb}$$

Cll at ℄

$$M_{℄} = \frac{15(60)}{4} = \underline{225,000} \text{ ft-lb}$$

Moment at ℄ $= \underline{\underline{922,000 \text{ ft-lb}}}$

$$Z = \frac{M}{S} = \frac{930,000(12)}{18,000} = 620 \text{ in.}^3 \qquad \underline{\underline{\text{Use 36 WF 182}}}$$

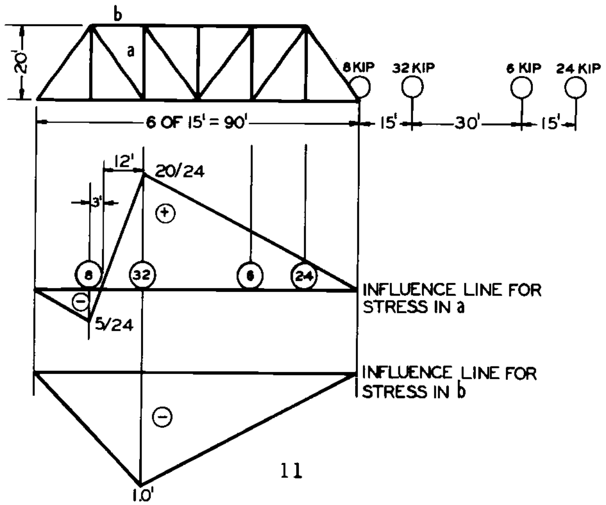
Influence lines for loads, stresses, deflection, and other items which vary with the position of loading, also can be drawn. It is common practice to draw influence lines for the axial loads in members of trusses subjected to moving loads.

Truss members are arranged and proportioned to resist the same type of internal loads imposed on beams. A truss is, in fact, nothing more than a deep beam with part of the web removed in order to give more efficient utilization of material.

The top and bottom chords (beam flanges) must resist the internal bending moments--the diagonal members must resist shear. It follows that the influence lines for top or bottom chord stresses will be of the same shape as an influence line for bending moment in a simple beam. Influence lines for stresses in diagonals will be similar to influence lines for shear in a simple beam.

Example: Stresses in Pratt Truss

The Pratt truss shown is to be designed for a stationary live load plus dead load of 600 lb/ft, plus the moving load shown. Determine the maximum stresses in members a and b when the live load moves across the bridge from right to left.



Influence line for stress in a:

$$\text{DL stress} = 0.6 \text{ kip/ft (area under IL)}$$

$$= 0.6\left[\frac{20}{24}\left(\frac{72}{2}\right) - \frac{5}{24}\left(\frac{18}{2}\right)\right] \qquad = 16.9 \text{ kip}$$

$$\text{LL stress} = \frac{20}{24}(32) + \frac{10}{24}(6) + \frac{5}{24}(24) - \frac{5}{24}(8) \qquad = \underline{32.5} \text{ kip}$$

$$\text{Total stress} \qquad = \underline{\underline{49.4 \text{ kip}}}$$

(tension)

Influence line for stress in b:

$$\text{DL} = 0.6(-1)\left(\frac{90}{2}\right) \qquad = -27.0 \text{ kip}$$

$$\text{LL} = -\frac{1}{2}(8) - 1\,(32) - \frac{1}{2}(6) - \frac{1}{4}(24) \qquad = -\underline{45.0} \text{ kip}$$

$$\text{Total stress} \qquad = \underline{\underline{-72.0 \text{ kip}}}$$

(compr)

Gravity dams may fail in a number of ways:

1. Overturning or tipping about the toe. The factor of safety against overturning is given as the ratio of righting moments to overturning moments.

2. Sliding of the dam relative to the foundation. The factor of safety against sliding is $FS = \mu\, \Sigma F_V / \Sigma F_H$, where μ is the coefficient of static friction between the dam and its foundation.

3. Foundation failure either through excessive pressures or through weakening of the foundation by excessive seepage.

4. Failure of material in the dam due to overstressing.

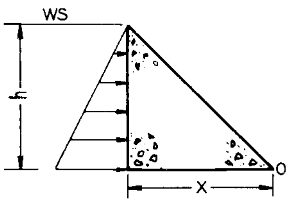
Example: Check of Dam Stability

Assume a triangular section, concrete gravity dam with vertical upstream face. Water surface is at the top of dam and there is no tailwater.

1. With zero uplift, what downstream slope would you provide if the dam is to be just on the verge of overturning?

2. With full uplift on the full area of the base, what downstream slope is required to be safe against overturning?

If the dam overturns it will pivot about point 0. It is on the verge of overturning when righting moments are equal to overturning moments.

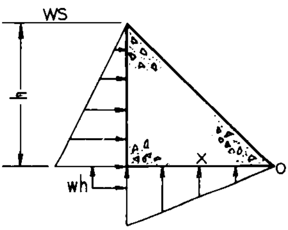


$$150\left(\frac{hx}{2}\right)\left(\frac{2}{3}x\right) = 62.4\,h\left(\frac{h}{2}\right)\left(\frac{h}{3}\right)$$

$$x^2 = \frac{62.4}{300}\,h^2$$

$$x = 0.456\,h$$

Downstream slope = $\underline{\underline{2.19}}$



$$150\left(\frac{hx}{2}\right)\left(\frac{2}{3}x\right) = 62.4\,h\left(\frac{h}{2}\right)\left(\frac{h}{3}\right) + \frac{62.4\,hx}{2}\left(\frac{2}{3}x\right)$$

$$50\,x^2 - 20.8\,x^2 = 10.4\,h^2$$

$$x^2 = 0.356\,h^2$$

$$x = 0.598\,h$$

Downstream slope = $\underline{\underline{1.67}}$

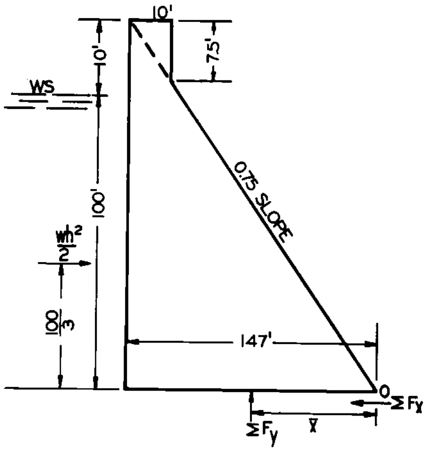
Example: Check of Dam Stability and Foundation

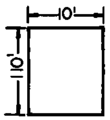
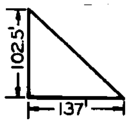
Given a concrete dam with water load as shown in the following sketch:

1. Determine foundation reaction at upstream and downstream faces and at a point 15 feet from the U.S. face. Disregard effect of uplift.

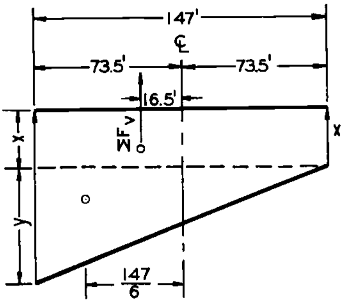
2. How much uplift, in terms of head of water, could occur at the upstream face before the stability of the structure against overturning is affected?

3. What is the shear safety factor (FS) if dam is founded upon andesite rock?



Unit	Volume (cu ft)	Force (kip)	Arm (ft)	Moment (kip-ft)
$\frac{wh^2}{2}$	5000	312	33.3	- 10,400
	1100	165	142.0	23,400
	7020	1053	91.3	96,300
		ΣF_V = 1218		ΣM_O = 109,300

$\bar{x}$ = 109,300/1218 = 90.0 ft



Check for foundation pressures:

$$1218 = \frac{x + y + x}{2}(147)$$

$$1218(16.5) = \frac{y}{2}(147)(\frac{147}{6})$$

$$y = 11.2 \text{ kip/ft}^2$$

$$x = \underline{\underline{2.7 \text{ kip/ft}^2 \text{ toe pressure}}}$$

$$x + y = \underline{\underline{13.9 \text{ kip/ft}^2 \text{ heel pressure}}}$$

$$13.9 - \frac{11.2}{147}(15) = \underline{\underline{12.8 \text{ kip/ft}^2}}$$

(Pressure 15 ft from U.S. face)

2. Assume triangular uplift pattern. Overturning moment of uplift must then balance net righting moment before stability of the structure is destroyed.

$$\frac{62.4 \, h \, (147)(91.3)}{2(1000)} = 109,300$$

$h = \underline{\underline{262 \text{ ft}}}$ (Uplift cannot overturn dam)

3. Assume $\mu = 0.75$. Shear FS (sliding) = 0.75(1218)/312 = $\underline{\underline{2.92}}$ if no uplift is considered.

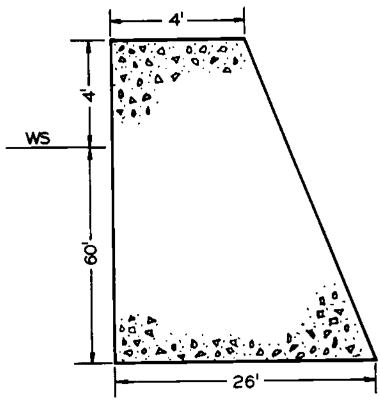
For full hydrostatic uplift

$$\Sigma F_V = 1218 - \frac{62.4(100)}{2000}(147) = 760 \text{ kip}$$

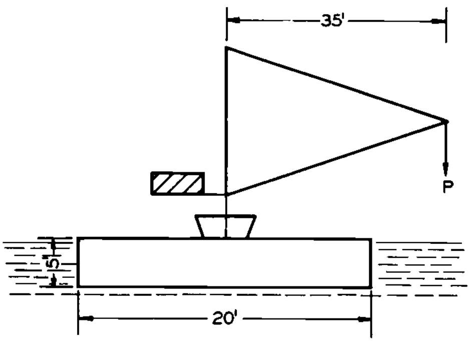
$$FS = \underline{\underline{1.83}}$$

Problems

1. A concrete dam of trapezoidal section as shown, has a vertical face 64 feet high. It is 4 feet wide at the top, 26 feet wide at the base, and weighs 150 pounds per cubic foot. If water stands with 4 feet of the top of the dam and the uplife coefficient is 0.3, varying uniformly along the bottom, determine the maximum and minimum intensities of pressure on the base. Would you consider this a good design for a gravity dam? If not, why not?

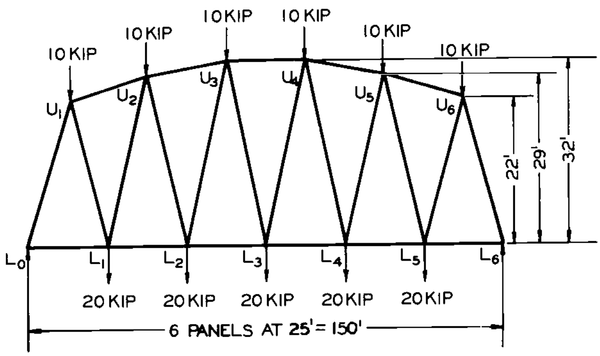


2. A barge is 20 feet wide and 50 feet long, displacing 100 tons, including the weight of the boom, counterweight, and tackle. What load can it pick up without the deck on the load side going under water? Answer: P = 9.75 kip.



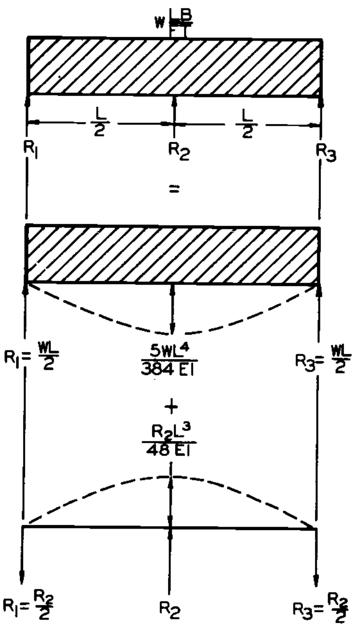
3. A weight of 3000 pounds is suspended by cables 7 feet, 8 feet, and 9 feet long from three points at equal elevation which form the points of an equilateral triangle whose sides are 10 feet in length. Neglecting cable weight, determine by either analytical or graphical methods the stress in the three tie cables due to the weight. Answer: 1900 lb, 1500 lb, 900 lb.

4. A six-panel camelback Warren truss has dead loads at upper and lower panel points, as shown. Find dead load stresses in members U_3 - U_4, U_3-L_3, and L_2 - U_3.



II. INDETERMINATE STRUCTURES

If a structure is so arranged and supported that it is not possible to analyze it by use of the equations of static equilibrium, the structure is said to be statically indeterminate. It is then necessary to seek additional equations or relationships for the solution of the problem from the geometry of the deflected structure; that is, expressions involving deflections and changes in slope of the elastic curve of the structure.



For the two-span continuous beam shown, note that it is statically indeterminate. If R_2 is removed, it is a simple beam, and the deflection of the elastic curve at that point can be determined.

If R_2 does not settle, the beam is not permitted to deflect at that point, and R_2 must be sufficiently large to counteract the deflection produced by the uniform load.

$$\frac{5\ wL^4}{384\ EI} = \frac{R_2 L^3}{48\ EI}$$

$$R_2 = \frac{5}{8}\ wL$$

and

$$R_1 = R_3 = \frac{wL}{2} - \frac{5}{16}\ wL$$

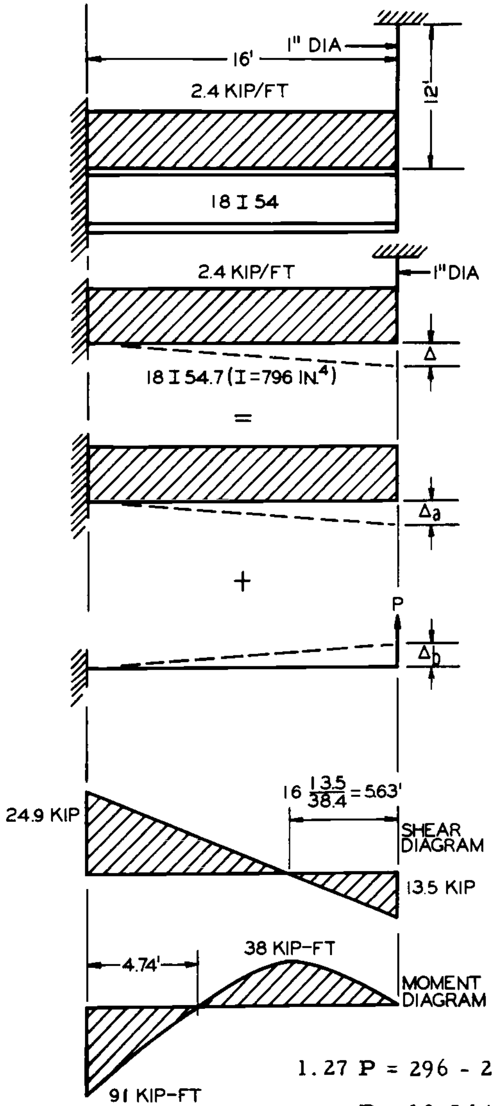
$$R_1 = R_3 = \frac{3}{16}\ wL$$

After the reactions have been determined, shear and bending moment diagrams can be plotted by conventional methods.

Example: Indeterminate I-Beam

An 18-inch I-beam at 54 pounds has one end fixed in a wall and the opposite end supported by a 1-inch round hanger rod. The

I-beam spans 16 feet and supports a uniform load of 2400 pounds per foot. Calculate the maximum stresses in the beam and rod. Design the end connection for the rod to the beam.



Neglect weight of beam

Δ = actual beam deflection
= rod elongation

$$\Delta = \frac{PL_R}{AE} \text{ for rod}$$

$$\Delta_a = \frac{WL^3}{8\ EI} = \text{beam deflection}$$

for uniform load only

$$\Delta_b = \frac{PL^3}{3\ EI} = \text{beam deflection}$$

for rod load only

$$\Delta = \Delta_a - \Delta_b$$

$$\frac{PL_R}{AE} = \frac{WL^3}{8\ EI} - \frac{PL^3}{3\ EI}$$

$$\frac{P(12)(12)}{0.785} = \frac{2.4(16)(16)^3(12)^3}{8(796)} - \frac{P(16)^3(12)^3}{3(796)}$$

$$1.27\ P = 296 - 20.6\ P$$

$$P = 13.5 \text{ kip}$$

$$S_{rod} = \frac{P}{A} = \frac{13,500}{0.785} = \underline{\underline{17,200 \text{ psi}}}$$

$$S_{beam} = \frac{Mc}{I} = \frac{91,000(12)(9)}{796}$$

$$S_{beam} = \underline{\underline{12,300 \text{ psi}}}$$

When it becomes necessary to determine deflections and changes in slope of the elastic curve of a structure, the moment-area method may be used. The moment-area approach is based on mathematical relationships correlating the bending moment, slope, and deflection diagrams. The two moment-area theorems may be stated as follows:

1. Change in slope of the tangents to the elastic curve between any two points a and b on the elastic curve, is given by the area under the bending moment diagram between a and b, divided by EI.

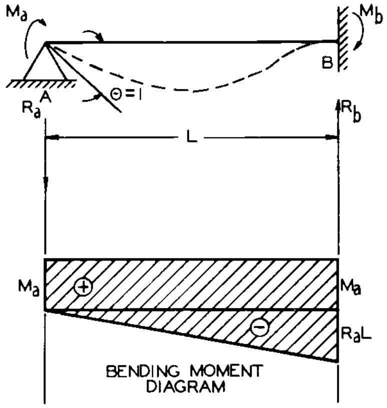
2. The deflection of point a on the elastic curve with respect to a tangent to the elastic curve at b, is given by the statical moment of that portion of the bending moment diagram between a and b about point b, divided by EI.

In order to illustrate the use of moment-area theorems, consider a beam (see sketch) simply supported at A and rigidly supported at B. Assume this beam is subjected to an applied moment at A which produces enough bending to rotate the tangent to the elastic curve at A through an angle of one radian.

The bending moment at any point from the left end of the beam is given as $M_a - R_a x$, and the diagram can be drawn in two parts, as shown.

Note that the tangent to the elastic curve at B must remain horizontal, and the deflection of the curve at A with respect to the tangent at B must be zero. Then the moment of the bending moment diagram about A must be zero.

$$\frac{M_a L}{EI}\left(\frac{L}{2}\right) - \frac{R_a L}{EI}\left(\frac{L}{2}\right)\left(\frac{2}{3} L\right) = 0$$



$$R_a = \frac{3}{2}\left(\frac{M_a}{L}\right)$$

$$M_b = M_a - \frac{3}{2}\left(\frac{M_a}{L}\right)(L) = \frac{1}{2} M_a$$

Note that the moment at B is 1/2 the moment at A and of different sign. This is the carryover moment for a beam of constant cross section.

In order to find the magnitude of M_a, note that the change in slope of the tangents to the elastic curve between A and B is $\theta = 1$ radian, which must be equal to the area under the moment diagram between these two points divided by EI.

$$\frac{M_a L}{EI} - \frac{R_a L}{EI}\left(\frac{L}{2}\right) = 1$$

$$\frac{M_a L}{EI} - \frac{3}{2}\left(\frac{M_a}{EI\,L}\right)\left(\frac{L^2}{2}\right) = 1$$

$$M_a = \frac{4\,EI}{L}$$

This is called the beam stiffness; that is, the externally applied moment necessary to rotate the free end of a propped cantilever beam through an angle of one radian. Note that the stiffness of the beam (resistance to rotation) increases as the modulus of elasticity and moment of inertia increase, and that it decreases as the length increases.

Carryover moment and beam stiffness as derived above are necessary tools in the use of the Hardy Cross method of moment distribution used in the analysis of indeterminate structures, which may be outlined as follows:

1. Assume all members to be fixed (locked) at each end and calculate the fixed end moments for the loadings given. Some of the joints will not be in equilibrium.

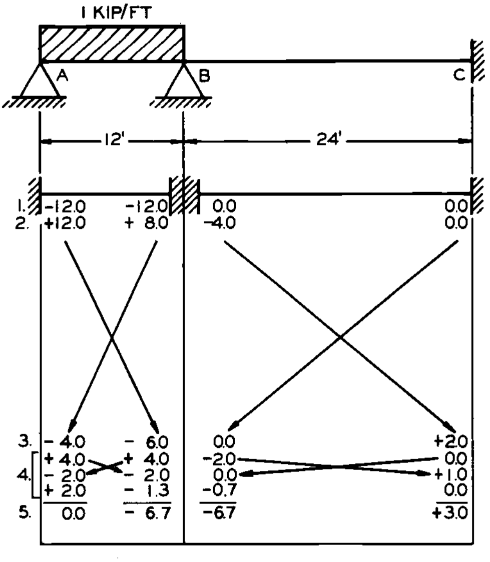
2. Unlock the joints and add or subtract the necessary moment to balance the joint. Stiffness of the members must be taken into account when distribution of the unbalanced moment is made.

3. Account for addition or subtraction of moment to one end of the member by crediting the carryover moment (one-half the magnitude and opposite sign) to the other end of the member. This will again throw some of the joints out of balance.

4. Repeat steps 2 and 3 until the carryover moments are small enough to be neglected in design.

5. Sum up of the increments of moment at each joint to obtain the moments to be used for design. Include the initial fixed end moments, all corrections, and carryover moments.

Assume the beam shown to be of constant cross section and find design moments at B and C.



Steps:

1. Lock joints and calculate fixed end moments.
2. Unlock joints and balance moments.
3. Apply carryover moments.
4. Balance, carryover, balance.
5. Sum increments of moment for design figure (ft-k).

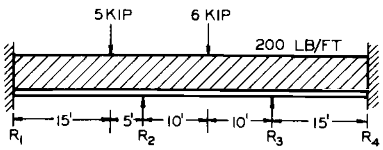
Note that on the actual structure the moment at A must be zero, the moments on each side of B must be the same, and the support at C can take any moment required of it. Because BC is twice the length of AB, it is half as stiff; therefore, twice as much unbalanced moment must be distributed to AB as to BC.

The foregoing analysis gives figures within approximately 10% of the true values. If greater accuracy is necessary, the solution can be carried through more steps. The sign convention used assigns positive to moments causing compression on the top fibers of the beam.

Example: Beam with Fixed Ends and Two Supports

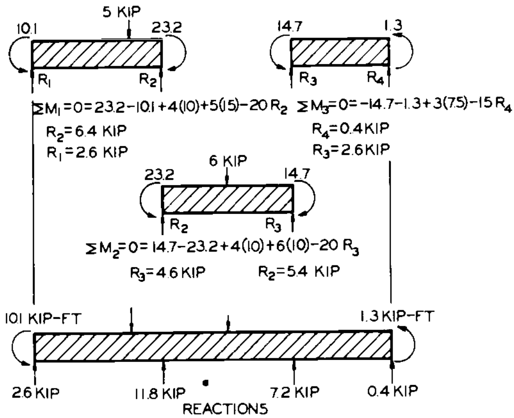
A beam has a uniform cross section throughout its length. It is fixed at both ends with simple intermediate supports and is loaded with a uniform load (dead load plus live load) of 200 lb per foot plus two concentrated loads.

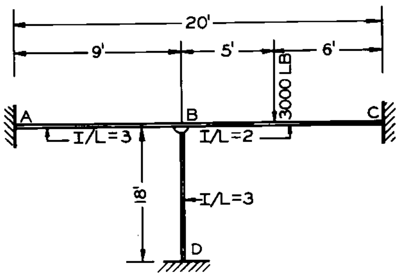
1. Find the moments at each of the four supports.
2. Find the reactions at each of the four supports.



I/20		I/20		I/15		I/L
	1/2	1/2	3/7	4/7		DISTR FACTOR
-6.7	-6.7	-6.7	-6.7	-3.8	-3.8	UNIFORM LOAD FEM= $wL^2/12$
-4.7	-14.1	-15.0	-15.0	0.0	0.0	CONC LOAD FEM= pab^2/L^2
-11.4	-20.8	-21.7	-21.7	-3.8	-3.8	TOTAL FEM
0.0	-0.5	+0.4	+7.7	-10.2	0.0	BALANCE
+0.3	0.0	-3.8	-0.2	0.0	+5.1	CARRY-OVER
0.0	-1.9	+1.9	+0.1	-0.1	0.0	BALANCE
+1.0	0.0	0.0	-1.0	0.0	0.0	CARRY-OVER
0.0	0.0	0.0	+0.4	-0.6	0.0	BALANCE
-10.1	-23.2	-23.2	-14.7	-14.7	+1.3	MOMENTS

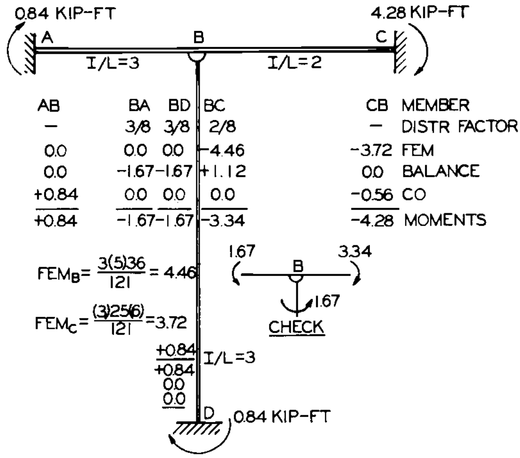
(Solution continued on next page)





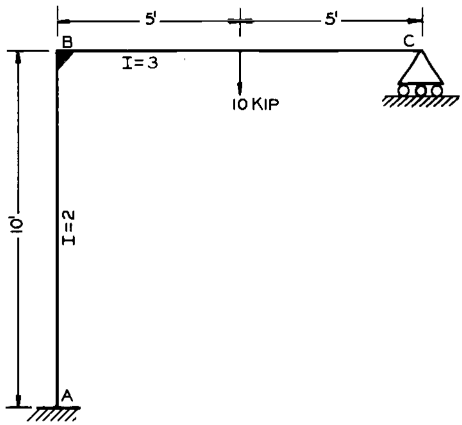
Example: Moment Distribution in Frame Joints

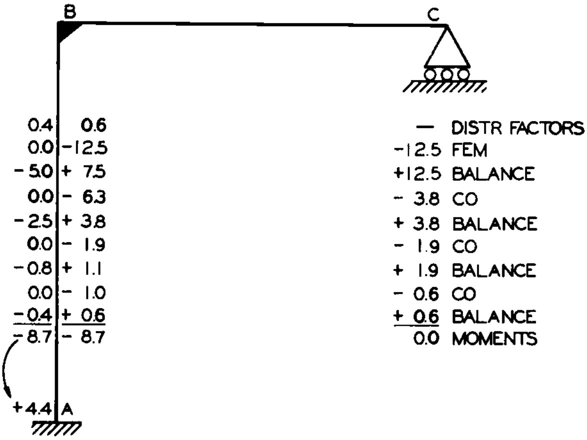
Determine by the Hardy Cross method of moment distribution the moments at all joints on the welded structural steel frame fixed at points A, C, and D, with the single concentrated load and properties of members as shown above.

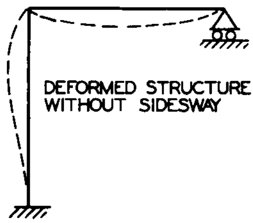


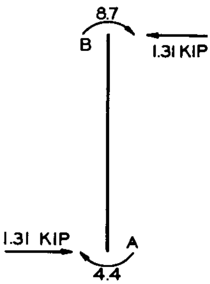
Example: Moments and Reactions in Frame

Calculate the moments and reactions in the rigid steel frame shown in the following sketch.

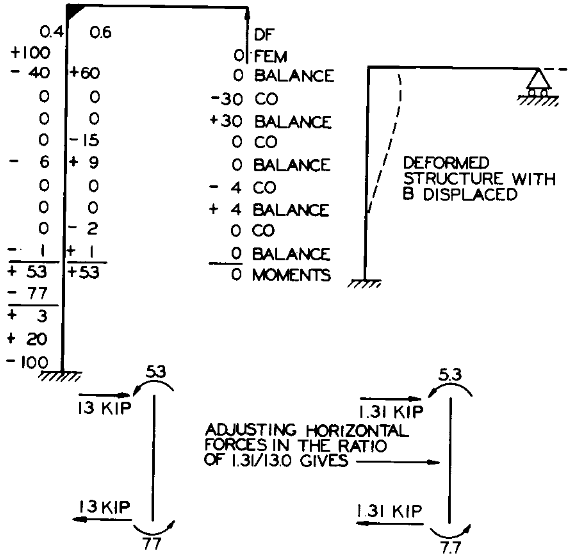




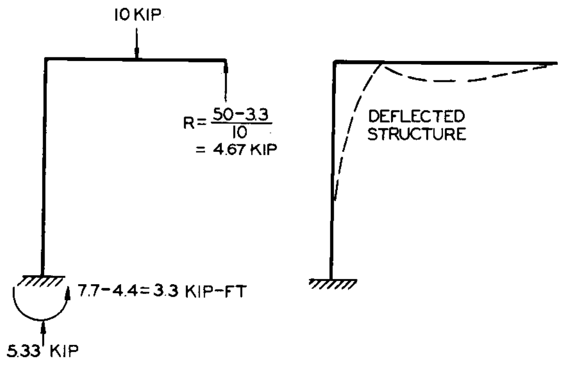




This distribution assumes that member B-C does not move horizontally. This develops a shear in leg A-B and a horizontal reaction of 1.31 kip at A. Because this is not possible with the loading shown, the sidesway of the structure must relieve this 1.31 kip thrust. In order to find the effect of sidesway, assume that joint at B is displaced horizontally without rotation enough to cause moments of 100 kip-ft at A and B, then release and distribute moments.

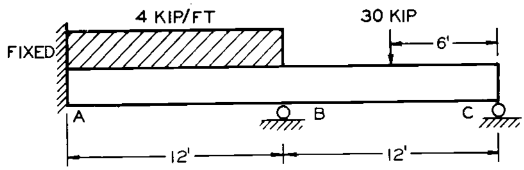


Combining the adjusted sidesway moments with the solution neglecting sidesway, the actual conditions are obtained.



Problem

A 12-inch WF 45 is loaded as shown in the diagram. Calculate reactions and draw shear and bending moment diagrams.



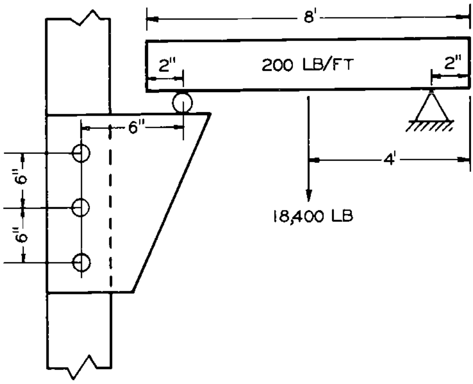
III. STRUCTURAL STEEL DESIGN

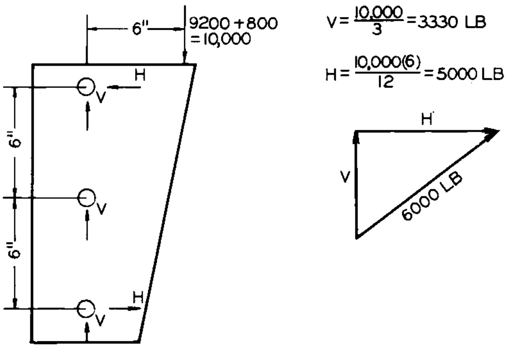
In the design and detailing of structural steel members, reference should be made to the Steel Construction Manual of the American Institute of Steel Construction. This handbook contains most of the necessary information for designing and detailing in accordance with accepted practice, together with much useful information of a more general nature. Much of the material intended for use in design is presented in time-saving tables and charts. Allowable stresses quoted in this section are from AISC Specifications unless otherwise noted.

Detail connections of structural steel members may be accomplished by riveting or welding. It is desirable that the connecting material (rivets or welds) be stressed in shear only, and that the use of this material in tension be reduced to a minimum. In design of connections subject to shear only, it is assumed that all the connecting material equally shares the load. If the connection is subject to a torque or moment because of eccentric loads, it is assumed the connecting material resists this torque unequally and the load assigned each increment of connecting material varies with distance from the center of gravity of the material.

Example: Shear Stress in Rivets

A bracket, which is riveted to a vertical channel with 7/8-inch rivets, is loaded as shown in the sketch. Determine the shear stress in each of the rivets. Use area of 7/8-inch rivet = 0.6 in.^{2}.



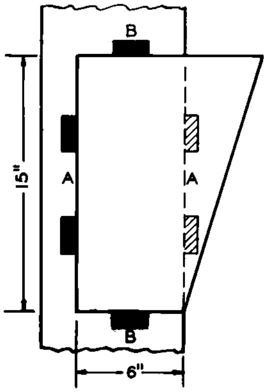


Top and bottom rivets: $S_s = \dfrac{6000}{0.6} = 10,000$ psi

Middle rivet: $S_s = \dfrac{3330}{0.6} = 5,550$ psi

If this connection was to be designed as a welded connection, welds A could be proportioned take the shear of 10,000 pounds, and welds B designed to resist the torque of 60,000 in-lb.

Based on a shearing stress of 13,600 psi specified by the American Welding Society, the resistance to shear of a fillet weld is $D = 0.707(13,600)(t) = 9600\ t$ lb/in. of weld, where t is the size of fillet weld.



For 1/4-inch fillet weld, the allowable load per inch of weld is 9600(1/4) = 2400 pounds.

Welds A: Length required = 10,000/4(2400) = 1.04 in./weld.

Welds B: Length required = 60,000/15(2400) = 1.67 in./weld.

Structural steel members can be classified for design purposes as:

1. Members subjected only to bending stresses (beams, girders).

2. Members subjected only to axial stresses (tension or compression members in trusses, bracing, columns, struts).

3. Members subjected to combined axial and bending stresses (beam columns, columns with eccentric loading).

Beams, girders, and other flexural members should be checked for possible failure, as follows:

1. Failure in bending. The extreme fiber stress (tension or compression) in bending should be computed from the flexure formula

$$S_b = \frac{Mc}{I}$$

where

S_b(lb/in.^{2}) is the fiber stress

M (in-lb) is the internal bending moment

C (in.) is the distance from the neutral axis to the outer fiber

I (in.^{4}) is moment of inertia about the neutral axis

The stress in bending should not exceed 20,000 psi, and in special cases (see below) this allowable stress must be reduced.

2. Failure due to lateral buckling of the compression flange. The compression flange of a flexural member may fail in buckling as a column. A criterion for lateral stiffness is the ratio Ld/bt, where

L (in.) is unsupported (laterally) length of compression flange
d (in.) is depth of beam
b (in.) is width of compression flange
t (in.) is thickness of compression flange

If the value of this ratio is less than 600, the allowable stress in bending for the compression flange is 20,000 psi. If the ratio exceeds 600, the allowable stress in bending for the compression flange is reduced to

$$S_b = \frac{12,000,000}{Ld/bt}$$

3. Failure in shear. The horizontal shearing stress at any point in a beam may be computed as

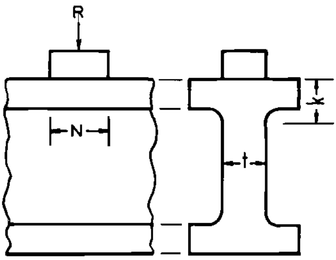
$$S_s = \frac{VQ}{It} = \frac{V(A\bar{y})}{It}$$

where

S_s (lb/in.^{2})	is horizontal shearing stress at point in question
V (lb)	is vertical shear (from shear diagram)
Q (in.^{3})	is statical moment of beam cross-sectional area above (or below) the neutral axis
I (in.^{4})	is moment of inertia about neutral axis
t (in.)	is thickness of beam web or width of beam flange, depending upon location of point in question

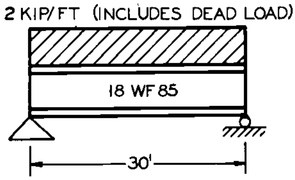
The horizontal shearing stress, which is a maximum at the neutral axis, should not exceed 13,000 psi.

4. Failure due to web crippling of beams. If a heavy load R is applied to a rolled section through a bearing of length N, as shown, the load must be carried in shear by the web. The minimum area resisting the load will be in the web at the toe of the fillets and is t (N + 2k).



The stress acting to cripple the web will be $S_c = R/t\ (N + 2k)$, which should not exceed 24,000 psi.

Investigate the strength of the beam shown.



$$M_{max} = \frac{wL^2}{8} = \frac{2(900)}{8} = 225 \text{ kip-ft}$$

$$V_{max} = \frac{wL}{2} = 30 \text{ kip}$$

1. Check bending

$$S_b = \frac{Mc}{I}$$

$$S_b = \frac{225(1000)(12)(9.15)}{1430}$$

$$= 17,300 \text{ psi versus } 20,000 \text{ allowable}$$

This beam is strong enough in pure bending, but

2. Check stiffness of compression flange

$$\frac{Ld}{bt} = \frac{360(18.3)}{8.84(0.911)} = 820$$

$$S_b = \frac{12,000,000}{820} = 14,700 \text{ psi allowable if the top flange is}$$

not laterally supported. If it is possible to support the top flange at midspan, the Ld/bt ratio is 410 and the allowable bending stress is raised to 20,000 psi. A stronger beam must be selected if it is not possible or practical to furnish lateral support.

3. Check horizontal shear at beam center.

$$S_s = \frac{VQ}{It}$$

$$S_s = \frac{30,000\left[0.911(8.84)(8.70) + 8.25(0.526)(4.13)\right]}{1430(0.526)}$$

$$S_s = 3520 \text{ psi versus } 13,000 \text{ allowable}$$

4. Check for web crippling at support.

$$S_c = \frac{R}{t(N + k)}$$

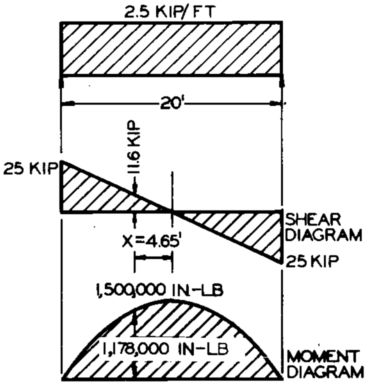
$$S_c = \frac{30,000}{0.526(3 + 1.5)}$$

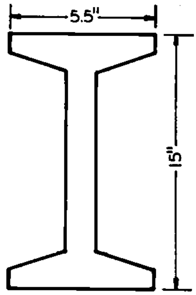
$$S_c = 12,700 \text{ psi versus } 24,000 \text{ allowable}$$

The beam is strong enough for the bearing provided, but practical considerations at the support may dictate more bearing.

Example: Design of Steel Beam

A 15-inch, 42.9-pound I-beam used on a span of 20 feet, center to center of bearings, is to be reinforced with flange plates to carry safely in flexure a total uniformly distributed load of 50,000 pounds. Make a sketch and give all design data.





15 I 42.9

$$I = 442 \text{ in.}^4, \ \frac{I}{c} = 58.9 \text{ in.}^3$$

Beam will carry $M = 58.9(20,000)$
$M = 1,178,000$ in-lb

Solve for distance x.

$$\frac{x}{10}(25)\frac{x}{2} = \frac{1,500,000 - 1,178,000}{12(1000)}$$

$$x = 4.65 \text{ ft}$$

Use two cover plates 10 feet long to care for additional moment. If plates work at 20,000 psi

$$27,000(12) = 20,000\, A_s(15.38)$$

$$A_s = 1.05 \text{ in.}^2$$

3/8 by 6 in. plate gives 2.22 in.^{2} <u>Use</u>

Moment of inertia of built-up section

$$I = 442 + 2(2.22)(7.69)^2$$

$$I = 704 \text{ in.}^4$$

Stress in bending

$$S_b = \frac{1,500,000(7.87)}{704} = 16,800 \text{ psi}$$

Check:

$$\frac{Ld}{bt} = \frac{240(15.75)}{6(3/8 + 5/8)} = 630$$

Allowable stress in bending = 12,000,000/630 = 19,000 psi <u>OK</u>

Design connection between 3/8 PL and I-beam.

Horizontal shear on top of I-beam $= S_s = \frac{VA\bar{y}}{It} = \frac{11,600(2.2)(7.6)}{704(5.5)}$

$$S_s = 50 \text{ psi}$$

Load tending to slip plate = 50(12)(5.5) = 3300 lb/ft. Five-eighths inch diameter rivet in single shear gives 4600 lb, so spacing could be 4600/3300(12) = 16.7 in., but AISC says 3/8(16) = 6 in. Fillet welds of 3/16 in. give 9600(3/16) = 1800 lb/in. Need 3300/1800 = 1.83 in. of weld per foot of beam.

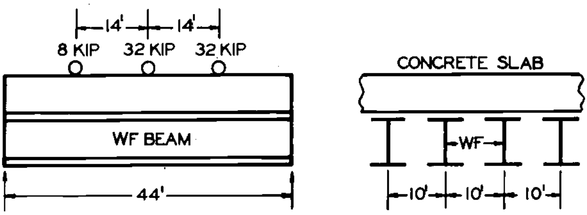
AISC { Maximum space between welds = 3/8(16) = 6 in.
Minimum length fillet weld = 1-1/2 in.

Use 2-inch welds at 8-inch spacing.

<u>Example: Design of Beams for Floor Support</u>

Architect's plans call for a truck freight unloading floor to have a clear span underneath of 44 feet. The floor is to be made of reinforced concrete slab on WF steel floor beams at 10 feet, center to center. The truck size is AASHO H-20-S-16. It may be assumed that each 44-foot beam must carry one truck for maximum moment plus the dead load. The WF beams are simply supported on concrete. Design the WF beam, and

1. Give maximum live load moment.
2. Select a wide flange beam and give actual shear and fiber stress in beam.



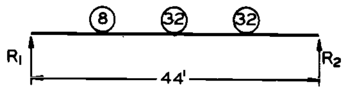
Design steel beams.

Dead load: Slab $= \frac{1}{2}(150)(10) = 750$ lb/ft

Assume beam at $\underline{250 \text{ lb/ft}}$

1000 lb/ft

Design for moment at ℄ of beam.



With 32-kip load at ℄,
$R_1 = 28.3$ kip, $R_2 = 43.7$,
$M_{\text{℄}} = 511$ kip-ft

$$M_{DL} = wL^2/8 = \frac{1(44)^2}{8} = 242$$

$$M_{LL} = \underline{511}$$

$$M_{\text{℄}} = 753 \text{ kip-ft}$$

$$\frac{I}{c} = \frac{753,000(12)}{20,000} = 452 \text{ in.}^3$$

Use 33 WF 141

Check stresses.

Absolute maximum moment comes when 32-kip load is 2.33 feet to left of beam centerline, then

$R_1 = 32.2$ $\qquad R_2 = 39.8$ $\qquad M = 521$

Adjusted DL moment $= 242(0.9) = \underline{218}$

Actual moment $= 739$

Stress in bending $= \dfrac{739,000(12)}{446.8}$

$$S_b = \underline{19,800 \text{ psi}}$$

Maximum shear when 32-kip load is over R_2

$$V = 32 + 32\left(\frac{30}{44}\right) + 8\left(\frac{16}{44}\right) + 0.900(22)$$

$$V = 76.5 \text{ kip}$$

Maximum horizontal shear

$$S_s = \frac{VA\bar{y}}{It}$$

$$S_s = 76,500\left[\frac{11.54(0.96)(16.17) + (15.69)(0.605)(7.85)}{7442(0.605)}\right]$$

$$S_s = \underline{\underline{4310 \text{ psi}}}$$

Structural steel members carrying only axial loads are subject to code restrictions in regard to stiffness as measured by the slenderness ratio (L/r) of the member, where

L is the unbraced length of the member
$r = \sqrt{I/A}$ is the radius of gyration of the member

The AISC Code requires that the slenderness ratio shall not exceed the following:

Main compression members---------------------------- 120
Bracing and other secondary members in compression----200
Main tension members---------------------------------- 240
Bracing and other secondary members in tension---------300

Certain exceptions to the foregoing values are permitted.

Tension members are proportioned for an allowable stress of 20,000 psi if the slenderness ratio falls within the specified limits.

Compression members exhibit a tendency to buckle and fail through column action before the P/A value reaches the allowable compressive stress of the material. The AISC Code permits the following P/A values for compression members:

$$\frac{P}{A} = 17,000 - 0.485(L/r)^2 \qquad \text{for values of } L/r \text{ under } 120$$

$$\frac{P}{A} = \frac{18,000}{1 + L^2/18,000\, r^2} \qquad \text{for values of } L/r \text{ between 120 and 200}$$

<u>Example: Column Design with Axial Loading</u>

An axially loaded column 21 feet long is subjected to a load of 90,000 pounds. Select a suitable wide flange section to carry this load.

Section	A	P/A	r	L/r	P/A allowable
8 WF 31	9.12	9880	2.01	125	9,640
8 WF 35	10.30	8750	2.03	124	9,710
10 WF 45	13.20	6810	2.00	126	9,560
10 WF 49	14.40	6250	2.54	99	12,250

If an L/r in excess of 120 is permissible, the 8 WF 35 is satisfactory. If not, the 10 WF 49 should be used.

Members subjected to both axial and bending stresses are proportioned so that

$$S_b / S_{b(allowable)} + \frac{P/A}{P/A(allowable)} \text{ does not exceed } 1.0.$$

The allowable stress in bending is taken as 20,000 psi unless the Ld/bt ratio exceeds 600. The allowable axial stress is computed by use of the column formulas previously given, and depends upon the slenderness ratio.

Example: Column Design with Eccentric Loading

Assume the 21-foot column of the previous problem is loaded eccentrically with 90,000 pounds 12 inches from the major axis. Select a suitable wide flange section.

Section	Allowable stress		Actual stress		$\frac{S_b}{S_{b(allow)}} + \frac{P/A}{P/A(allow)}$
	Bending	Axial	$S_b = M_c/I$	P/A	
10 WF 49	20,000	12,250	19,800	6250	0.99 + 0.51 = 1.50
10 WF 77	20,000	12,400	12,500	3960	0.63 + 0.32 = 0.95
12 WF 65	20,000	13,600	12,300	4700	0.62 + 0.35 = 0.97
14 WF 61	20,000	11,900	11,700	5020	0.59 + 0.42 = 1.01

Check Ld/bt for 14 WF 61

$$\frac{Ld}{bt} = \frac{21(12)(13.9)}{10(0.643)} = 545 \qquad \underline{\underline{\text{Use}}}$$

IV. REINFORCED CONCRETE DESIGN

Structural members are frequently composed of different materials working together to carry the imposed loads. The key to analysis of such composite members lies in observing that if the two materials are to work together in resisting loads, then the unit strains at points of connection must be the same for either material.

$$\epsilon_1 = \epsilon_2$$

$$S_1/E_1 = S_2/E_2$$

$$S_1 = S_2(E_1/E_2) = nS_2$$

which states that the unit stresses in the two materials are related by the modulus of elasticity ratio, n. The cross-sectional area of material necessary to carry some imposed load P, would be

$$P = S_1A_1 \qquad\qquad P = S_2A_2$$

$$A_1 = P/S_1 = P/nS_2 \qquad\qquad A_2 = P/S_2$$

$$nA_1 = P/S_2 \qquad = \qquad A_2 = P/S_2$$

Thus an area A_2 may be replaced by an equivalent area nA_1 for purposes of analysis.

Example: Reinforced Wooden Beam

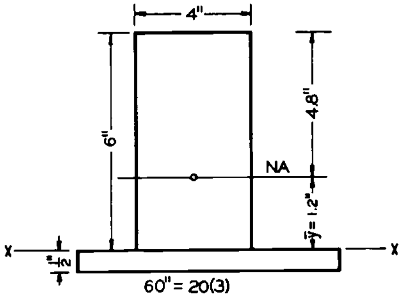
A 3-inch by 1/2-inch steel strap is securely fastened to the underside of a Douglas fir beam 4 inches wide and 6 inches deep. What uniformly distributed load will this beam sustain safely on a simple span of 16 feet?

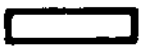
Assume: $E_S = 30,000,000$ psi

$E_W = 1,500,000$ psi

$E_S/E_W = 20$

EQUIVALENT WOOD SECTION



Shape	Area	y	Ay	Ay^2	I_{g-g}	I_{x-x}
	24	3	72	216	72	288
	30	-1/4	-7.5	2	--	2
	54		64.5			290

$$\bar{y} = 64.5/54 = 1.2 \text{ in.}$$

$$I_{NA} = 290 - 54(1.2)^2 = 212 \text{ in.}^4$$

Assume working stress in wood = 1400 psi.

$$\text{Allowable moment} = M = \frac{SI}{C} = \frac{1400(212)}{4.8}$$

$$M = 62,000 \text{ in-lb}$$

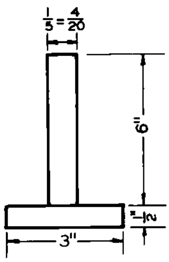
For uniform load

$$M = \frac{wL^2}{8}$$

$$\frac{62,000}{12} = \frac{w(16)^2}{8}$$

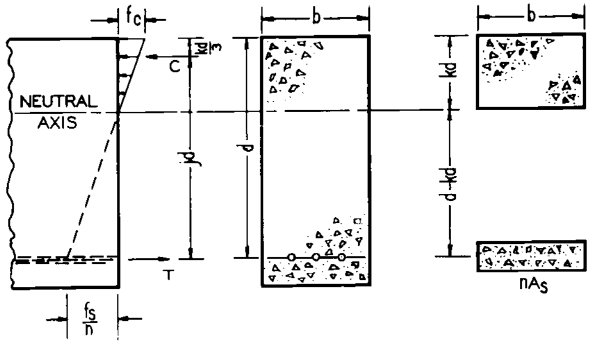
$$w = \underline{\underline{161 \text{ lb/ft}}}$$

The problem could be analyzed in a similar manner by assuming an equivalent steel section.



The analysis of reinforced concrete members is based on the relationships outlined and on the following assumptions regarding the action of the materials under load:

1. The concrete carries no tensile stress.
2. Plane sections before bending remain plane after bending.



(a) (b) (c)

Figures a, b, and c above are definitive sketches showing conventional notations used in reinforced concrete design. Figure a shows straight line stress distribution assumed for a beam subjected to a bending moment which puts compression on the top fibers. The total compressive load carried by the concrete is

$$C = \frac{1}{2} f_c \, b(kd)$$

which acts at the centroid of the stress diagram, or kd/3 from the top of the beam. The total tensile load carried by the steel is

$$T = f_s A_s$$

and is assumed to act at the centroid of the steel area.

The resisting moment the beam can supply is

$$M_c = C(jd) = \frac{1}{2} f_c \, b(kd)(jd)$$

or

$$M_s = T(jd) = f_s A_s (jd)$$

depending upon which equation gives the lower value when the allowable stresses for concrete and steel are substituted in the above equations. If both concrete and steel stresses reach the allowable at the same time, the beam is said to have balanced design.

The values of j and k for balanced design vary within narrow limits, depending upon allowable concrete and steel stresses, but for design purposes values of $j = 0.875$ and $k = 0.375$ may be used without appreciable error.

Example: Design of Reinforced Concrete Floor Slab

Architect's plans call for a truck freight unloading floor to have a clear span underneath of 44 feet. The floor is to be made of reinforced concrete slab on WF steel floor beams at 10 feet, center to center. The truck size is AASHO H-20-S-16. It may be assumed that each 44-foot beam must carry one truck for maximum moment plus the dead load. The WF beams are simply supported on concrete. Use ACI specifications with $f'_c = 2500$ psi. Design floor slab, assuming it is continuous over several beams. Use AASHO specifications for floor slab moment.

Assume:

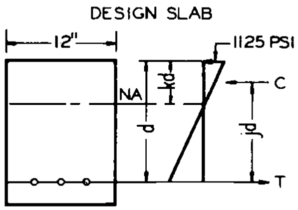
S = 10 ft

E = width of slab over which load is distributed

E = 0.4 S + 3.75 = 7.75 ft (AASHO)

$$BM = \pm 0.2(P_1/E)S = \pm 0.2(12,000/7.75)(10)$$

$$= \pm 3100 \text{ ft-lb/ft (AASHO)}$$



$$f_s = 20,000, \; f_c = 1125, \; kd = \frac{3}{8} d,$$

$$jd = \frac{7}{8} d$$

$$3100(12) = 12(3/8 \; d)(1125/2)(7/8 \; d)$$

$d = 4.1$ in. Use 6-in. slab

$$T = C = 12(\frac{3}{8})(4.1)(\frac{1125}{2}) = 10,400$$

$$A_s = 10,400/20,000 = 0.5 \text{ in.}^2$$

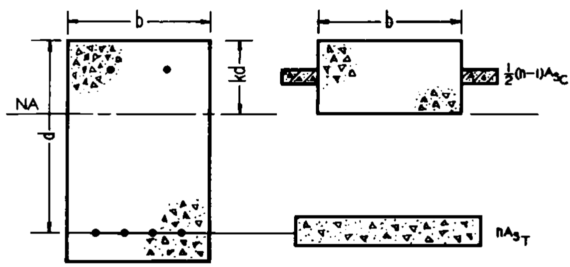
Use 5/8-in. diameter bars at 7-in. spacing perpendicular to traffic.

Parallel to traffic: percentage $A_s = 100/\sqrt{S} = 100/\sqrt{10} = 31.5\%$

0.16 in.^{2} use 3/8 diameter bars at 8-in. spacing

(AASHO)

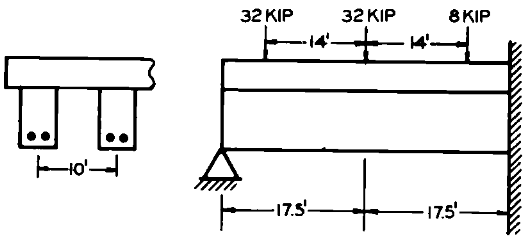
When reinforced concrete beams are subjected to large bending moments, it may be necessary to add compressive steel to the top flange. The transformed (equivalent concrete) section of such a beam is shown below.



The compression steel would then be stressed to nf_c, where f_c is the stress in the concrete at the same distance from the neutral axis as the centroid of the compressive steel.

Example: Reinforced Concrete Beam-Slab Floor System

A floor system consists of 35-foot span reinforced concrete beams simply supported at one end and fixed at the other. The beams are 10 feet, center to center. The slab must be designed for a live load of 65 pounds per square foot. Each beam will carry the concentrated live loads shown. $f'_c = 3000$ psi, $f_s = 20,000$ psi. Will a beam 16 inches wide and with a stem of $d = 36$ inches be satisfactory for this design? If not, what size would you use?



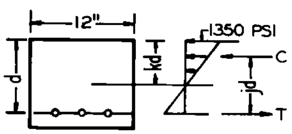
Design slab in order to determine dead load (assume 4 in.).

$w = 65 + 50 = 115 \text{ lb/ft}^2$ $\qquad L = 10 - 1.33 = 8.67$ ft

$$M = \frac{wL^2}{10} = \frac{115(8.67)^2}{10} = 865 \text{ ft-lb}$$

Assume $k = 0.375$, $j = 0.875$

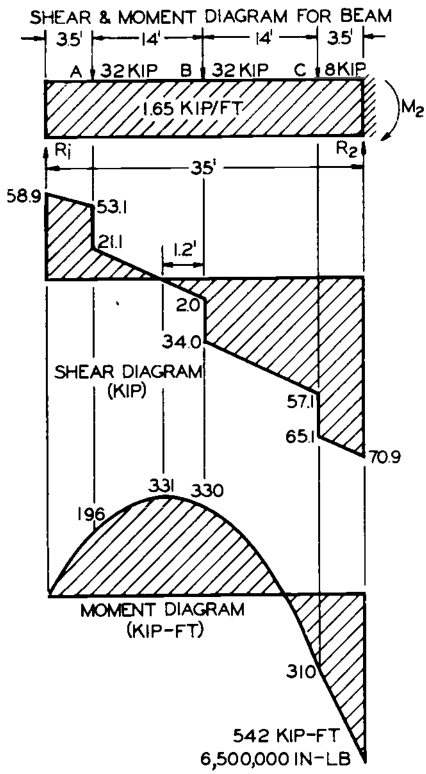
$M = 865(12) = 12(0.375d)(1350/2)(0.875d)$



$d = 1.97$ in.

$A_s = M/f_s(jd) = 0.30 \text{ in.}^2$

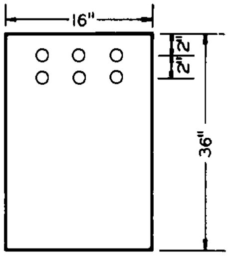
Use wire mesh. 4-inch slab OK.



Solve for R_1, R_2, M_2 from AISC beam formulas:

Loading	R_1	R_2	M_2
UDL 1 kip/ft	13.1	21.9	153
ULL 0.65 kip/ft	8.5	14.3	100
CLL (A)	27.2	4.8	55
CLL (B)	10.0	22.0	210
CLL (C)	0.1	7.9	24
Total	58.9	70.9	542
	kip	kip	kip-ft

Because we have negative moment at the wall, the slab gives no help in bending. For balanced design, assuming two rows of tension steel:



$d = 33$ in.
$kd = 0.403\ d = 13.3$ in.
$jd = 0.866\ d = 28.6$ in.
$M = 16(13.3)(1350/2)(28.6)$
$M = 4{,}100{,}000$ in-lb

$$A_s = \frac{4{,}100{,}000}{20{,}000(28.6)} = 7.16 \text{ in.}^2$$

Compression steel must be provided to take care of additional moment of

$$6{,}500{,}000 - 4{,}100{,}000 = 2{,}400{,}000 \text{ in-lb}$$

Compression steel works at a stress of

$$(10)\ 1350(11.3/13.3) = 11{,}500 \text{ psi}$$

on an arm of 33 - 3 = 30 inches of two rows of composition steel.

$$\text{Compression steel} = \frac{2{,}400{,}000}{11{,}500(30)(9/10)} = 7.75 \text{ in.}^2$$

$$\text{Additional tension steel} = \frac{2{,}400{,}000}{20{,}000(30)} = 4.0 \text{ in.}^2$$

(Note: All the steel needed cannot be placed in 16-in. width. Suggest haunched beams 24 in. by 48 in. at support.)

The bond stress acting between the concrete and steel is given as

$$u = \frac{V}{\Sigma o(jd)}$$

where V is the shear in the beam as taken from the shear diagram, and Σo is the perimeter of reinforcing steel in contact with concrete at the section where V is taken. Bond stresses may be reduced by using smaller bars in greater numbers for a required A_s.

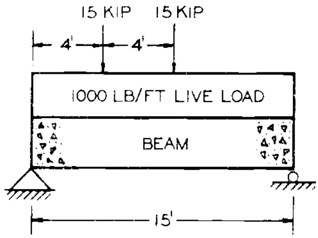
The maximum shearing stress in the concrete at any point between the neutral axis and the tensile reinforcement is given as

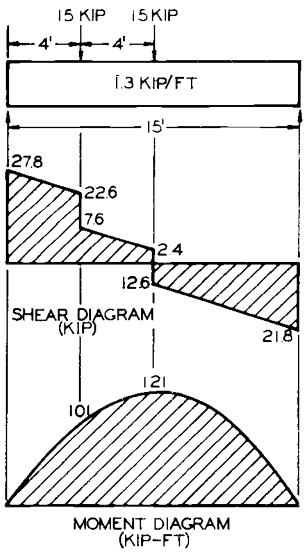
$$S_s = \frac{V}{b(jd)}$$

Web reinforcement in the form of stirrups or bent up bars must be provided to carry shear in excess of the allowable for the concrete.

Example: Reinforced Concrete Beam with Simple Supports

Design a simply supported reinforced concrete beam to have a span of 15 feet and to carry live loads, as shown. Use 18,000 lb/sq in. for steel and 2500 lb/sq in. for concrete.



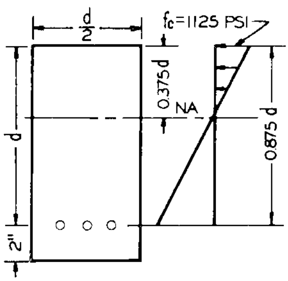


Assume beam weighs 300 lb/ft

$b = d/2$

$k = 0.375$

$j = 0.875$



$$M = 121(12)(1000) = \frac{d}{2}(0.375\ d)\left(\frac{1125}{2}\right)(0.875\ d)$$

$$d^3 = 15{,}700$$

$$d = 25 \text{ in.}$$

Use $d = 25$ in.

$b = 12$ in.

(Beam good for 122.5 kip-ft using balanced design - Seelye)

$$M = A_s f_s (jd)$$

$$A_s = \frac{121(12)(1000)}{18{,}000(0.875)(25)} = 3.68 \text{ in.}^2$$

Use three No. 10 bars ($A_s = 3.80$ in.^{2})

Check bond:

$$u = \frac{V}{\Sigma o(jd)} = \frac{27{,}800}{3(3.99)(0.875)(25)} = 106 \text{ versus 250 allowable. OK.}$$

Check shear:

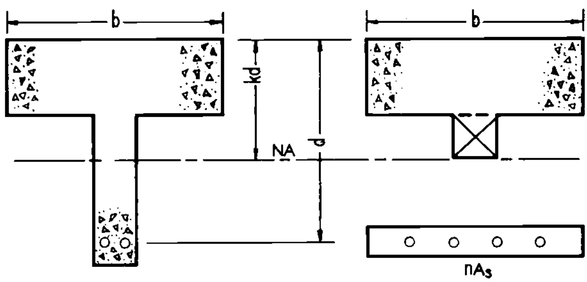
$V_c = vb(jd) = 75(12)(0.875)(25) = 19.7$ kip carried by concrete.

Assume 3/8-in. diameter stirrups and check for spacing:

$$S = A_v f_v (jd)/V' = \frac{2(0.11)(18{,}000)(0.875)(25)}{27{,}800 - 19{,}700} = 10.7 \text{ in. at left support}$$

Maximum spacing = $d/2$ = 12.5 in.

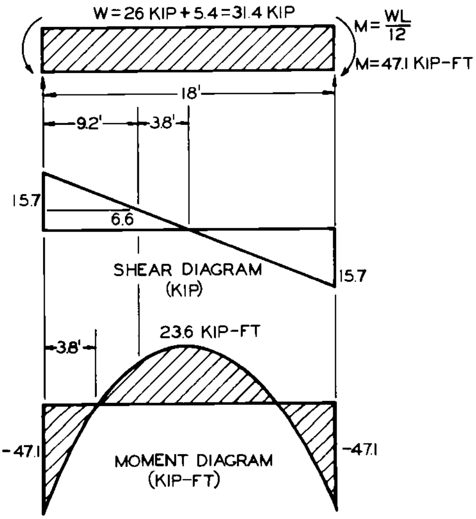
The transformed section for a concrete T-beam is shown below.



For convenience in analysis, the concrete below the flange and above the neutral axis is considered to be of no value in resisting bending moment. It should be noted that, as the proportions of the beam change, the NA may move up so that only part of the flange is in compression.

Example: Reinforced Concrete Beam with Restrained Ends

Design a reinforced concrete T-beam having a span of 18 feet between faces of supports and supporting a uniformly distributed load of 26,000 pounds, exclusive of the weight of the beam. Ends are fully restrained.



Estimate weight of beam at 300 lb/ft.

Assume $f'_c = 2500$ psi

$n = 12$

$f_s = 20,000$ psi

Invert T-beam since max moment is negative at supports.

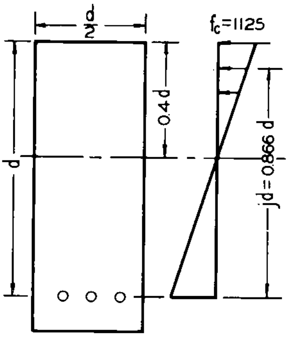
Proportion stem for moment at center of span with $b = d/2$.

$$M = 23,600(12) = \frac{d}{2}(0.4\,d)(0.866d)(1125/2)$$

$$d^3 = 2900$$

$$d = 14.3 \text{ in.}$$

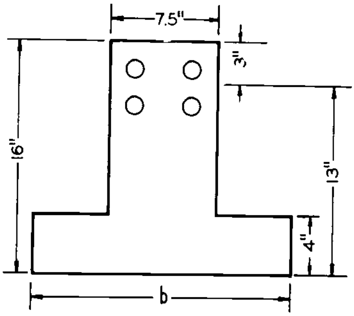
Use $b = 7.5$ in. $\qquad d = 14$ in.



$$A_s = \frac{23,600(12)}{20,000(0.866)(14)} = 1.17 \text{ in.}^2$$

Use two No. 7 bars.

FOR ⊥ AT SUPPORT



Flange depth > 3.75 in.
Flange width < 30 in. } ACI code

Assume flange depth = 4 in.
Assume 2 rows of steel.
Assume jd = 0.9 d = 11.7 in.

$$A_s = \frac{47,100(12)}{20,000(11.7)} = 2.41 \text{ in.}^2$$

Use 4 No. 7 bars.

Assume kd = 0.4 d = 5.2 in.
C = T = 2.41(20,000) = 4b (690)
b = 17.5 in. Use b = 18 in.

$$f_c\text{avg} = \frac{1125 + 1.2/5.2(1125)}{2}$$
$$= 690 \text{ psi}$$

Check shear:

$$v = \frac{V}{b(jd)} = \frac{15,700}{7.5(11.7)} = 179 \text{ psi} \qquad \text{Use stirrups.}$$

Shear carried by concrete = V_c = 75(7.5)(11.7) = 6600 lb

Stirrup spacing at support = S = $A_v F_v (jd)/V'$

Assume 3/8-in. diameter stirrups, $S = \dfrac{0.22(20,000)(11.7)}{9100}$

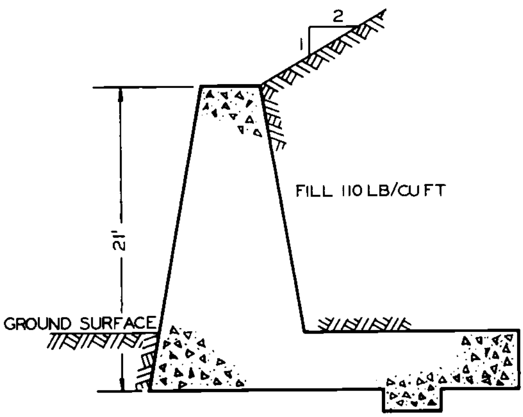
$= 5.65$ in.

Maximum stirrup spacing $= \frac{d + 1}{2} = 7$ in.

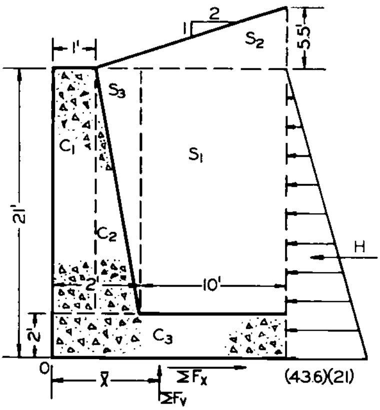
Retaining walls of reinforced concrete are designed as flexural members, with the bending moments at the base of the stem and the forward edge of the base setting the required dimensions from the standpoint of strength in bending. In addition to the strength of the wall alone, further investigation must be made as to its stability against sliding or overturning, and as to magnitude and distribution of soil pressures on the stem and base.

Example: Retaining Wall Design

A retaining wall with a cross section similar to that shown is to be designed. The wall will be 21 feet high from the base. The surcharge slope is 2-1. The wall will be on the property line, therefore it must be cantilevered. Counterforts will not be used. The allowable soil-bearing pressure is 2-1/2 tons per sq ft, and the weight of the fill back of the wall is 110 lb per cu ft. Use these allowable stresses: $f'_c = 2500$ psi, $f_s = 20,000$, and ACI specifications. Coefficient of friction between subsoil and concrete base is 0.5. Select dimensions for wall and show reinforcing.



Based on a given unit of weight, bearing pressure, and surcharge slope, assume an equivalent fluid pressure, w' = 30 lb/cu ft (Seelye). Assume following dimensions for retaining wall:



Adjust equivalent fluid weight for surcharge.

$$w' = 30\left[2-\left(\frac{21-5.5}{21}\right)^2\right] = 43.6 \text{ lb/ft}^3$$

$$H = \frac{43.6(21)}{2}(21) = 9600 \text{ lb}$$

Find position of resultant on base.

Unit	Volume (cu ft)	Force (kip)	Arm (ft)	Moment about 0 (kip-ft)
C_1	19.0	2.85	0.50	+ 1.4
C_2	9.5	1.42	1.33	+ 1.9
C_3	24.0	3.60	6.00	+ 21.6
S_1	190.0	20.90	7.00	+ 146.5
S_2	30.2	3.33	8.33	+ 27.7
S_3	9.5	1.05	1.67	+ 1.7
H		9.60	7.00	- 67.3
	ΣF_V =	33.15 kip	ΣM_o =	+ 133.5 kip-ft

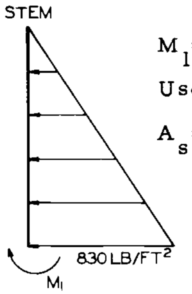
$\bar{x} = \frac{133.5}{33.15} = 4.02$. Good, gives triangular soil loading on base.

Max soil pressure at 0 = $\frac{33.15}{6} = 5.53$ kip/ft^2 versus 5.00 allowable.

Check against sliding:

Max friction = 33.15(0.5) = 16.6 kip versus 9.6 kip. OK.

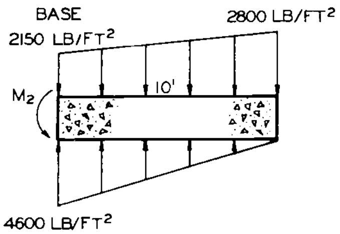
Design: Use tables for balanced design (Seelye, pp 1-35).



$M_1 = 50$ kip-ft

Use d = 16 in.

$A_s = 2.18$ in.^{2}

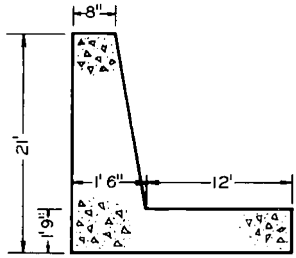


$M_2 = 67.5$ kip-ft

Use d = 19 in.

$A_s = 2.58$ in.^{2}

Final dimensions: This section requires 1.28 cu yd/ft of wall as compared to 1.95 for previous design.



$\Sigma F_V = 37$ kip, $\Sigma M_o = 173.3$ kip-ft

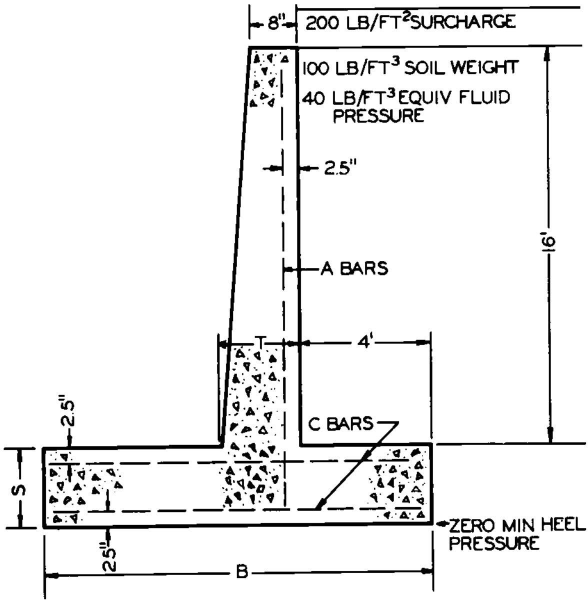
$\bar{x} = 4.7$ ft OK.

Max soil pressure still slightly over allowable, but should be OK.

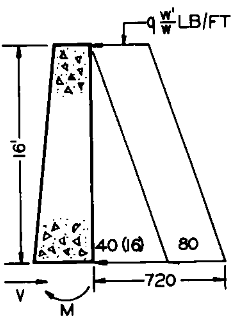
<u>Example: Retaining Wall Design (more detailed)</u>

The retaining wall shown maintains a coefficient of sliding friction of 0.5 or less. Assume 2000-lb concrete and reinforcing steel with allowable 18,000 lb/sq in. working stress, in accordance with ACI-318-51 code, and determine:

1. Required wall thickness, T.
2. Required base slab thickness, S.
3. Size and spacing of A bars.
4. Size and spacing of C bars.
5. Width of base B.



Load on stem. (Refer Seelye for data on surcharge.)



$q(\frac{w'}{w})$ lb/ft = $200(\frac{40}{100}) = 80$ lb/ft increase in pressure due to surcharge.

$$V = \frac{640}{2}(16) + 80(16) = 6400$$

$$M = 5120(\frac{16}{3}) + 1280(\frac{16}{2}) = 37,600$$

For balanced design use beam tables (Seelye, p 134).

$d = 15.5$, $A_s = 1.68(\frac{20,000}{18,000}) = 1.87 \text{ in.}^2$

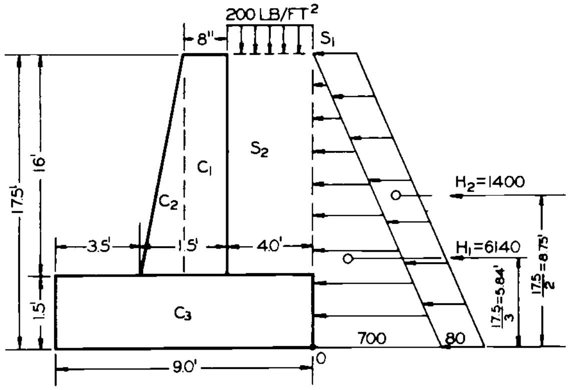
$T = 15.5 + 2.5 = 18$ in.

No. 10 A bars at 8-in. spacing. Cut alternate bars 8 ft from top.

Stability of wall. Assume S = 18 in., B = 9 ft.

$$H_1 = 700/2(17.5) = 6140 \text{ lb}$$

$$H_2 = 80(17.5) = 1400 \text{ lb}$$



Unit	Volume	Force	Arm	Moment
C_1	10.7	1.60	4.33	6.93
C_2	6.7	1.00	4.95	4.95
C_3	13.5	2.03	4.50	9.14
S_1	8.0	0.80	2.00	1.60
S_2	64.0	6.40	2.00	12.80
H_1	--	6.14	5.84	35.80
H_2	--	1.40	8.75	12.27
	$\Sigma F_V =$	11.83	$\Sigma M_o =$	83.49
	$\Sigma F_H =$	7.54		

$$\bar{x} = \frac{83.5}{11.8} = 7.08 \text{ ft}$$

$$\mu = \frac{7.54}{11.83} = 0.64$$

Too high, too far.

Redesign base. S = 18 in., B = 11 ft. Extend heel 2 feet to the right.

Horizontal forces remain the same. Additional vertical forces:

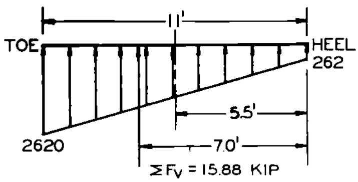
Surcharge, 2(200)	= 400 lb
Soil above heel, 2(16)(100)	= 3200 lb
Added concrete in base, 2(225)	= 450
	4050 lb
	= 4.05 kip

$$\text{New } \Sigma F_V = 11.83 + 4.05 = 15.88 \text{ kip}$$

$$\text{New } \Sigma M_O = 83.5 + 2(11.83) + 1(4.05) = 111.20 \text{ kip-ft}$$

$$\text{New } \bar{x} = \frac{111.20}{15.88} = 7.0 \text{ ft} \qquad \text{OK}$$

$$\mu = \frac{7.54}{15.88} = 0.475 \qquad \text{OK}$$



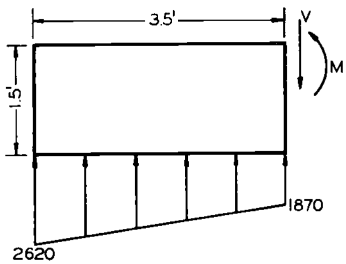
$$P = \frac{\Sigma F_V}{B}\left(1 \pm \frac{6e}{B}\right)$$

$$P = \frac{15,880}{11}\left[1 \pm \frac{6(1.5)}{11}\right]$$

$$P = 1442(1 \pm 0.818)$$

$$P = 2620 \text{ lb/ft}^2 \text{ at toe}$$

$$P = 262 \text{ lb/ft}^2 \text{ at heel}$$



Check toe for bending.

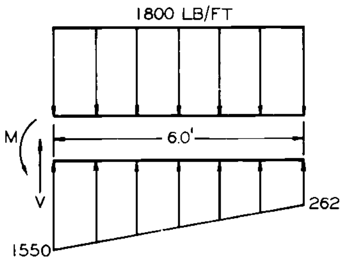
$$M = 1870(3.5)\left(\frac{3.5}{2}\right) + 750\left(\frac{3.5}{2}\right)\left(\frac{2}{3}\right)(3.5) - 225(3.5)\left(\frac{3.5}{2}\right)$$

$$= 13.130 \text{ ft-lb}$$

With $d = 15.5$, $jd = 13.6$

$$A_s = \frac{13,130(12)}{18,000(13.6)} = 0.65 \text{ sq in.}$$

Use No. 6 bars at 8 inches.



Check heel for bending.

$$M = 1800(6)(3) + 225(6)(3)$$

$$- 262(6)(3) - 1288(\frac{6}{2})(\frac{6}{3})$$

$$M = 24,000 \text{ ft-lb}$$

$$A_s = \frac{24,000(12)}{18,000(13.6)}$$

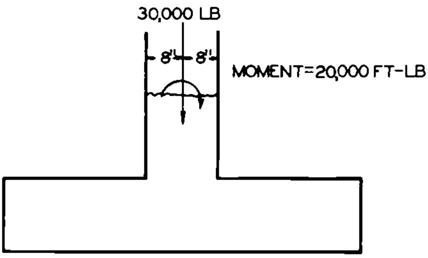
$$A_s = 1.18 \text{ sq in.}$$

Use No. 8 bars at 8 inches.

Problems

1. A precast, concrete pile wall is to be driven for the purposes of furnishing a flood wall 6 feet high and a cut-off wall extending 20 feet into the ground. The piles are to be 8 inches thick by 24 inches wide, by 26 feet long, and will provide 1 foot of freeboard over the maximum flood of record. For the purpose of exact determination, assume the pile is firmly held at the ground line and that the reinforcement required to resist driving stresses will be 1-1/2% of the cross-sectional area. Determine the additional steel, if any, required to resist the hydraulic stress.

2. Design an eccentrically loaded reinforced concrete footing for a rigid frame. The direct load is 30,000 lb per foot of wall, and the moment from the wall at the footing is 20,000 ft-lb per foot of wall. Use an allowable soil bearing pressure of 4000 lb per sq ft, 18,000-lb steel, and 2500-lb concrete. Show a detail of the size of the footing, including all reinforcing.



3. A reinforced concrete cantilever retaining wall is required which would be 15 feet high from the top of the heel slab to the top of the wall. Fill material to be placed behind it weighs 100 lb/cu ft and has a 30-degree natural angle of repose. Design heel and toe slab dimensions and thickness of stem, indicating position of resultant pressure. Indicate position, spacing, and size of reinforcement in stem and slab.

4. From calculations made for a rectangular reinforced concrete pressure conduit, the moment and direct stress at a certain point in one of the walls were found to be: M = 27 kip-feet per foot, and direct stress = 10 kips per foot tension. By the transformed area method determine the stresses in the steel and concrete at this point. Total thickness of wall = 18 inches. Reinforcement consists of 1-inch diameter bars spaced at 6 inches on tension side. Distance from concrete face to ℄ reinforcement on tension side = 3 inches, n = 10. Kd was found to be 4.31 inches.

V. HYDRAULICS

Hydraulics problems discussed in this section will be restricted to steady flow conditions where the volume rate of flow (Q cfs) is equal to the product of the cross-sectional area of the stream (A sq ft), and the average velocity (V ft per sec) perpendicular to the cross-sectional area at any section

$$Q = A_1V_1 = A_2V_2 = A_3V_3$$

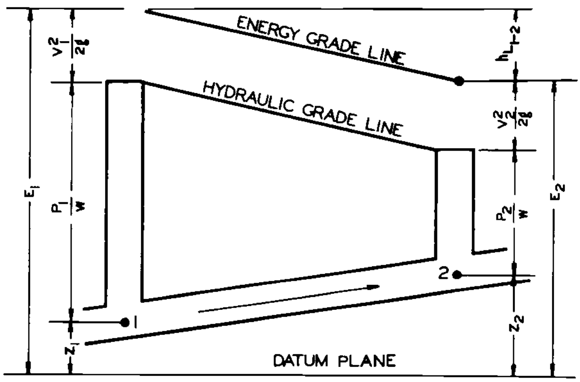
The energy equation for hydraulic analysis may be written

$$E_1 = E_2 + \text{energy loss}_{1-2}$$

or

$$\frac{V_1^2}{2g} + \frac{P_1}{w} + Z_1 = \frac{V_2^2}{2g} + \frac{P_2}{w} + Z_2 + h_{L_{1-2}}$$

This may be shown in graphical form for a pipe flowing full.



The energy grade line (EGL) and the hydraulic grade line (HGL) show the value of energy head and piezometric head for any point in the system. The energy head is defined as the sum of velocity head ($V^2/2g$), pressure head (P/w), and elevation head (Z). The piezometric head is defined as the sum of pressure head and elevation head. By this definition the hydraulic grade line must always lie below the energy grade line a distance $V^2/2g$, and must be parallel to the energy grade line in regions of uniform flow (no change in velocity). Note the dimensions of the term <u>head</u> are energy per pound of fluid, ft-lb/lb, or simply ft.

The energy loss term h_L, due in this case to pipe friction, must be evaluated through the use of experimental data. Many formulas, all of them using experimental data, have been proposed. The most generally applicable formula is the Darcy-Weisbach formula for head loss due to pipe friction,

$$h_L = f \frac{L}{D} \frac{V^2}{2g} \qquad \text{for pipes}$$

or

$$h_L = f \frac{L}{4R} \frac{V^2}{2g} \qquad \text{for noncircular conduits}$$

where f, the friction factor, is an experimentally determined coefficient which varies with the interior roughness of the pipe and the Reynolds number of the flow. In most civil engineering design application, the Reynolds number is high, and f varies only with the roughness of the pipe.

The hydraulic radius R is defined as the ratio of cross-sectional area of flow to length of wetted perimeter P of conduit. For circular pipes flowing full, $R = D/4$.

In addition to the Darcy-Weisbach formula, two other proposed formulas have been generally accepted and widely used:

$$V = 1.318\, C_{HW} R^{0.63} S^{0.54} \qquad \text{(Hazen-Williams)}$$

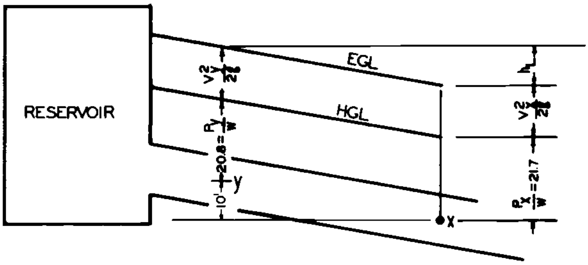
$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \qquad \text{(Manning)}$$

where the Hazen-Williams coefficient C_{HW}, and the Manning roughness factor n are again experimentally determined coefficients which depend upon the roughness of the channel lining. $S = h_L/L$ is defined as the slope of the energy grade line.

<u>Example: Flow Analysis</u>

A 30-inch pipe line from a reservoir was discharging

36 cfs. The discharge was reduced suddenly to 18 cfs, with no apparent change in conditions of the pipe. It was suspected that a timber had been carried down from the reservoir and lodged at a bend in the pipe at point x. It was known that the pipe was clear at point y. The pipe was tapped and pressure was measured at the two points. At y it was found to be 9.0 pounds per square inch. At x, which was 10 feet lower in elevation and 1000 feet distant, the pressure was 9.4 pounds per square inch. The coefficient n for the pipe is 0.016. Show by mathematical analysis if an obstruction exists.



$R = D/4 = 0.625$

$R^{2/3} = 0.732$

$V_y = 18/4.91 = 3.67$

$\frac{V_y^2}{2g} = 0.21$

$\frac{P_y}{w} = 9(2.31) = 20.8$

$\frac{P_x}{w} = 9.4(2.31) = 21.7$

The Manning formula can be written:

$$S = \left[\frac{Vn}{1.486\ R^{2/3}}\right]^2 = \left[\frac{3.67(0.016)}{1.486(0.732)}\right]^2$$

$S = 0.0029$

$h_L = 2.9$ ft

$E_y = E_x + h_L$

$$0.21 + 20.8 + 10 = \frac{V_x^2}{2g} + 21.7 + 2.9$$

$$\frac{V_x^2}{2g} = 6.41 > 0.21$$

The foregoing value of $V_x^2/2g$ offers the following possibilities:

1. The timber is lodged at x, in which case the velocity of

flow through the constricted area must increase.

2. The timber is lodged between y and x, in which case the computed value of h_L is much lower than the true value.

Losses, in addition to pipe friction losses, may be expected at any time the fluid velocity is changed. These are called minor losses and occur at conduit contractions, expansions, and bends. Pipe fittings or imperfectly aligned joints also cause additional losses. Valves, in effect, are adjustable head loss producers. The magnitude of any of these minor losses can be expressed generally as

$$h_L = K \frac{V^2}{2g}$$

where K is determined from experimental data (available in civil engineering or hydraulics handbooks).

<u>Example: Determination of Conduit Size</u>

An outlet conduit through a dam has the following characteristics:

1. Streamlined entrance.
2. Slide gate controlled 20 feet from upstream face.
3. Rectangular outlet throughout.
4. Passes horizontally through dam at elevation 100 feet.
5. Headwater at elevation 200 feet.
6. Length of conduit 50 feet.

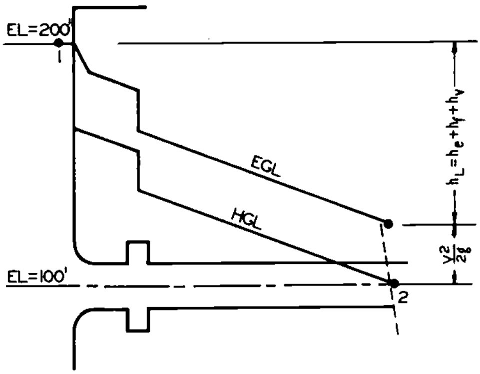
Find the size to discharge 500 cfs and plot pressure gradient.

Assume:

$f = 0.015$, use $h_f = \frac{fL}{D} \frac{V^2}{2g}$

$K_e = 0.05$, loss coefficient for entrance

$K_v = 0.20$, loss coefficient for gate in open position



$$E_1 = E_2 + \text{loss}$$

$$200 = 100 + \frac{V^2}{2g} + 0.05\,\frac{V^2}{2g} + \frac{15}{1000}\,\frac{50}{4R}\,\frac{V^2}{2g} + 0.2\,\frac{V^2}{2g}$$

Before the above equation can be solved, it is necessary to assume dimensions for the conduit. Try 3 ft by 3 ft, R = 0.75.

$$100 = \frac{V^2}{2g}(1 + 0.05 + 0.25 + 0.2)$$

$$\frac{V^2}{2g} = \frac{100}{1.5} = 66.7 \text{ ft}$$

$$V = 65.5 \text{ ft/sec}$$

Required: $$A = \frac{500}{65.5} = 7.65 \text{ sq ft}$$

which is smaller than the assumed 9 sq ft of area. Dimensions can be reduced to give approximately 8 sq ft of area, if desired.

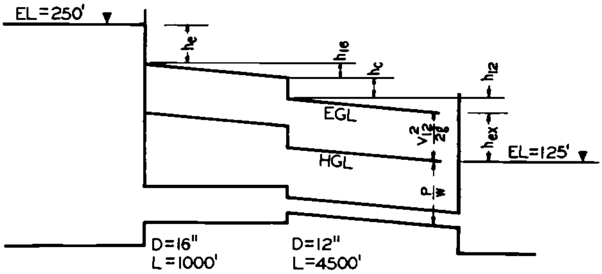
For purposes of plotting the pressure grade line, the following data may be calculated:

Entrance loss	= 0.05(66.7) =	3.3 ft
Valve loss	= 0.20(66.7) =	13.3
Friction loss	= 0.25(66.7) =	16.7
Exit vel head	= 1.00(66.7) =	66.7
Total		100.0 ft Check.

Slope of pressure grade line = 16.7/50 = 0.334.

Example: Flow Between Reservoirs

Two reservoirs, with water surface elevations 250 and 125 feet, respectively, are connected by a compound pipe line consisting of 1000 feet of 16-inch pipe followed by 4500 feet of 12-inch pipe. The point of transition from 16 inches to 12 inches is at elevation 215 feet. What is the flow through the line?



Assume:

$$f = 20/1000$$
$$K_e = 0.50$$
$$K_c = 0.10$$
$$K_{ex} = 1.00$$
$$E_1 = E_2 + h_L$$

$$250 = 125 + 0.5\left(\frac{V_{16}^2}{2g}\right) + \frac{20}{1000}\left(\frac{1000}{1.33}\right)\left(\frac{V_{16}^2}{2g}\right) + 0.1\left(\frac{V_{12}^2}{2g}\right)$$

$$+ \frac{20}{1000}\left(\frac{4500}{1}\right)\left(\frac{V_{12}^2}{2g}\right) + 1.00\left(\frac{V_{12}^2}{2g}\right)$$

but

$$Q = A_{16}V_{16} = A_{12}V_{12}$$

and

$$\frac{V_{16}^2}{2g} = \left(\frac{12}{16}\right)^4 \left(\frac{V_{12}^2}{2g}\right) = 0.316\left(\frac{V_{12}^2}{2g}\right)$$

$$125 = \frac{V_{12}^2}{2g}\left[0.5(0.316) + \frac{20}{1.33}(0.316) + 0.1 + 90 + 1.0)\right]$$

$$125 = \frac{V_{12}^2}{2g}(0.16 + 4.75 + 0.1 + 90 + 1)$$

$$\frac{V_{12}^2}{2g} = \frac{125}{96} = 1.30$$

$$V_{12} = 9.15 \text{ ft/sec}$$

$$Q = \underline{\underline{7.18 \text{ cfs}}}$$

Note in this case that the losses due to minor losses (entrance, pipe contraction, exit) total only $1.26(V_{12}^2/2g)$, as compared to a friction loss of $94.75(V_{12}^2/2g)$, or 1.3 per cent of the total loss. If these losses are neglected entirely, a variation in flow of only 0.7 per cent is evident. This variation is not significant in light of the probable error in the assumed friction factor.

As a rule of thumb in deciding whether or not to consider minor losses, the following classification is useful:

1. Long pipe--$L/D > 1000$; minor losses may be neglected.
2. Short pipe--$L/D < 1000$; minor losses must be considered.

Most practical problems in civil engineering design are concerned with turbulent flow patterns where the value of the friction factor is determined by the pipe roughness alone. At lower Reynolds numbers, however, the viscosity of the fluid also influences the value of the friction factor, and in the laminar flow range the friction factor varies with Reynolds number only.

$$f = \frac{64}{N_R}, \text{ which holds only when } N_R < 2100$$

The Reynolds number is a dimensionless ratio which is characteristic of a given flow pattern in the pipe.

$$N_R = \frac{VD}{\nu}$$

where

N_R = Reynolds number
V = average velocity in pipe (ft/sec)
D = diameter of pipe (ft)
ν = kinematic viscosity of fluid (sq ft/sec)

Example: Power to Pump Fuel Oil

A heavy fuel oil having a specific gravity of 0.915 and a kinematic viscosity of 0.00312 sq ft/sec at 50°F, is to be pumped through a 12-inch steel pipeline. Assuming temperature to remain constant, what horsepower per mile of pipe will be required to overcome friction in the pipe when the mean velocity of the oil is 4 feet per second?

$$N_R = \frac{4(1)}{0.00312} = 1280$$

$$f = \frac{64}{N_R} = 0.050$$

$$h_f/\text{mile} = f\left(\frac{L}{D}\right)\left(\frac{V^2}{2g}\right) = \frac{50}{1000}\left(\frac{5280}{1}\right)(0.25) = 66$$

$$hp = \frac{QwH}{550} = \frac{3.14(62.4)(0.915)(66)}{550} = \underline{\underline{21.5 \text{ hp}}}$$

The Darcy-Weisbach equation may be written in the form

$$h_f = rQ^2, \text{ or } Q = \sqrt{h_f/r}$$

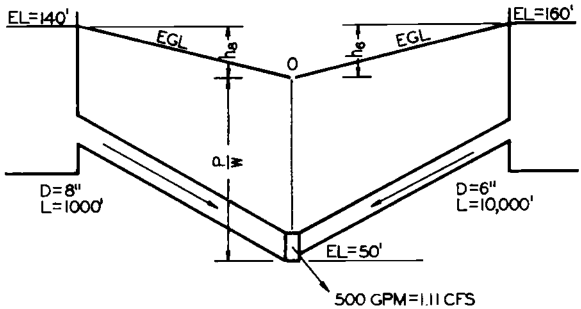
where

$$r = f\left(\frac{L}{D}\right)\left(\frac{1}{A^2 2g}\right) = \text{constant } f\left(\frac{L}{D^5}\right)$$

This is a convenient form when repeated use of the equation is necessary for a given problem. A convenient yardstick in the use of the above equation is the fact that if $L = 1000$ ft, $D = 1$ ft, and $f = 0.020$, then $r = 0.5$. This permits rapid calculation of the r value for a given pipe.

Example: Gravity Flow Through Two Feeder Lines

A fire hydrant at elevation 50 feet is supplied through 10,000 feet of 6-inch pipe from a reservoir at elevation 160 feet, and through 1000 feet of 8-inch pipe from a reservoir at elevation 140 feet. The hydrant is opened so the discharge is 500 gallons per minute. What is the rate of flow through each of the two pipes, and what is the residual pressure in pounds per square inch at the hydrant (assuming clean cast-iron pipe), and disregarding all losses except friction? Assume f = 0.023.



For the 8-inch pipe: $r_8 = 0.5\left(\frac{0.023}{0.020}\right)\left(\frac{1000}{1000}\right)\left(\frac{12}{8}\right)^5 = 4.4$

For the 6-inch pipe: $r_6 = 0.5\left(\frac{0.023}{0.020}\right)\left(\frac{10,000}{1000}\right)\left(\frac{12}{6}\right)^5 = 184$

This problem can be solved algebraically by writing

$$Q_8 + Q_6 = 1.1$$

$$r_6 Q_6^2 - r_8 Q_8^2 = 20$$

and solving the two equations simultaneously.

A simpler solution is obtained by assuming an elevation for point 0, solving for the flows, and checking to see that the flows into and out of the hydrant balance.

El point 0	h_8	h_6	Q_8	Q_6	Q_H
137	3	23	0.828	0.352	$1.18 \neq 1.11$
138	2	22	0.675	0.345	$1.02 \neq 1.11$

From the trials above, the proper elevation for 0 is approximately 137.5.

$$\text{Residual pressure} = 137.5 - 50 = 87.5 \text{ ft water}$$
$$= \underline{\underline{38 \text{ psi}}}$$

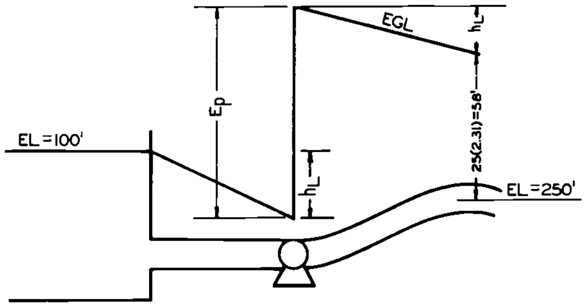
The same type of solution can be carried out using a nomograph for the Hazen-Williams formula (Seelye, pp 6-63), C = 120.

El point 0	$h_8/1000$	$h_6/1000$	Q_8	Q_6	Q_H
133	7	2.7	550	150	700
135	5	2.5	455	145	600
137	3	2.3	360	140	550

A pump placed in fluid system adds energy to the fluid. In order to decide upon the necessary size of pump and motor and to estimate operating costs, it is necessary to determine the system requirements.

Example: Pumping Costs

Water from a reservoir having a surface at a constant elevation of 100 feet is pumped over a hill having a summit elevation of 250 feet. The centerline of the pump is at elevation 90 feet, and when the pump is running a gage on the pipe line at the summit of the hill reads 25 pounds per square inch pressure. It has been determined that the total loss of head from friction and other causes between the reservoir and the summit of the hill is 45 feet. The pumping unit under head was found to have an overall efficiency of 55%. What will be the annual saving if a new pumping unit is installed having an overall efficiency of 65%? Electric power costs 1¢ per kwhr and the average daily pumpage is 1,000,000 gallons. Q = 1.55 cfs.



For this system the pump must lift each pound of water 150 feet in elevation, supply it with 58 feet of pressure head, and add sufficient energy to overcome the losses in the system (45 ft).

$$E_p = 150 + 58 + 45 = 253 \text{ ft-lb/lb}$$

The power which must be added to the water is

$$hp = \frac{QWE_p}{550} = \frac{1.55(62.4)(253)}{550} = 44.6 \text{ hp}$$

The power which must be purchased for a unit efficiency of 55% is

$$hp = \frac{44.6}{0.55} = 81.0 \text{ hp}$$

The required power for a unit efficiency of 65% is

$$hp = \frac{44.6}{0.65} = 68.5 \text{ hp}$$

Increased efficiency of the pumping unit results in a continuous saving of 12.5 hp.

$$\text{Annual saving} = (12.5)(0.746)(24)(365)\left(\frac{1}{100}\right) = \underline{\underline{\$820 \text{ per year}}}$$

Since the head losses in any system vary as the square of the flow rate, it follows that as the flow through a system changes, the head which the pump adds must change also. It becomes necessary to match the pump performance data with the system-head curve in order to assure satisfactory operation of both pump and system. If more than one pump is involved, a combined head-discharge curve for the multiple pump installation can be constructed.

This curve will take one shape if the pumps are operated in series and another if they are operated in parallel.

Example: Pump Performance Versus System Head Characteristics

For many years a town pumped its water with two centrifugal pumps A and B; running them in parallel during times of high demand. When increased demand necessitated more capacity, the town officials bought a used centrifugal pump of larger capacity to replace pump A. It was found after installation, however, that although pump C would pump more water than pump B when pumping alone, B and C together would pump little more than A and B had pumped together. The town officials, believing the pump defective, have engaged you to investigate the situation.

You find the setup is such that a rough calibration of the pumps is relatively simple. Accordingly, you test the pumps to determine the head discharge relationships. You also determine the head loss characteristics of the pump suction and discharge lines. The assembled data is as follows:

Pumps				Pipe line	
Head, ft	Discharge, gpm			Head loss, ft	Discharge, gpm
	A	B	C		
150	0	0	0	3.9	100
140	0	56	0	13.2	200
130	40	100	0	27.9	300
120	73	133	107	48.0	400
110	98	157	177	72.0	500
100	117	174	224		
90	130	188	257		

1. Explain the reason for the unsatisfactory operation of pumps B and C together.

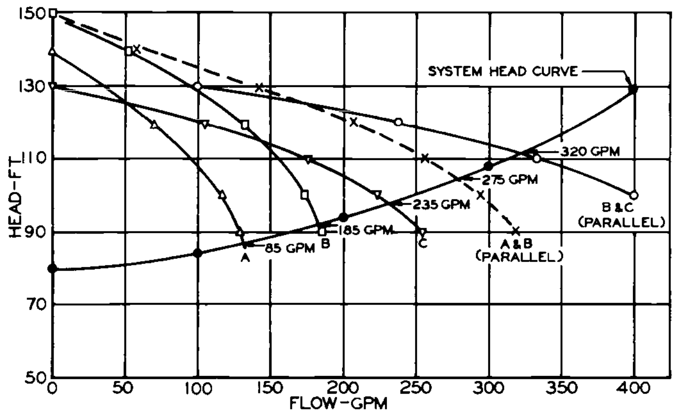
2. What quantities can be delivered at the same elevation at a pressure of 35 lb/sq in. by

a) A alone
b) B alone
c) C alone
d) A and B together
e) B and C together

Solution:

1. As flow increases, system head curve rises and pumps produce less flow.

2. See values below.



The head-discharge curve for a given pump is obtained from test data taken with the pump turning at some constant speed throughout its operating range. Pump performance at other speeds may be determined by testing at other speeds, or may be forecast in accordance with the following rules:

Q varies with rpm

H varies with $(\text{rpm})^2$

hp varies with $(\text{rpm})^3$

The above relationships are based on the assumption that the pump efficiency does not change at a given point of operation. They can be expected to hold for moderate speed changes, but should be used with caution.

Example: Flow Rate, Head, Power, and rpm Relationships

Given: Case I. A centrifugal pump at point A pumping through a pipeline and discharging at point B. The head loss in the pipe,

(h_1) is 40 feet, and the difference in elevation (h) between the pump discharge and point B is 100 feet.

Case II. A second pipe of the same size and kind is laid parallel to the first and the same quantity is now pumped through the two pipes as was discharged through the single pipe in Case I.

Required: 1. If 70 hp is required to discharge a given quantity of water through the pipe in Case I, what hp is required in Case II?

2. In Case I, if the pipe is straight, smooth, 12-inch diameter pipe and the quantity discharged is 3.93 cubic feet per second, what is the approximate length of the pipe?

3. In Case I, if the quantity delivered at point B is 1000 gpm and the speed of the pump is 875 rpm, what would be the theoretical capacity and total dynamic head developed by this pump if the speed is increased to 1750 rpm?

4. In Case I, if the pump is pumping 1000 gpm of brine with a specific gravity of 1.2, what is the efficiency of the pump if 70 bhp is required?

Solution: 1. $h_L = rQ^2$. If flow through one pipe is cut in half, then the new friction head is $40(1/2)^2 = 10$ feet, and the new pump head is 110 feet.

$$\text{hp} = 70\left(\frac{110}{140}\right) = \underline{\underline{55 \text{ hp}}}$$

2. Q = 3.93 cfs = 1770 gpm. Hazen-Williams formula (C = 120) from nomograph

$h/1000 = 7.8$ ft $\qquad L = \frac{40}{7.8}(1000) = \underline{\underline{5130 \text{ ft}}}$

Darcy-Weisbach formula (f = 0.020)

$\frac{V^2}{2g} = 0.39 \qquad L = \frac{40(1)}{0.020(0.39)} = \underline{\underline{5140 \text{ ft}}}$

3. $H_1 = 140$ ft

$H_2 = 140\left(\frac{1750}{875}\right)^2 = 560$ ft, which allows a new friction head of $\underline{\underline{460 \text{ feet.}}}$

Since the flow varies with the square root of h_f

$$Q_2 = 1000\sqrt{460/40} = \underline{\underline{3400 \text{ gpm}}}$$

(The above assumes a flat head-discharge curve on the pump and no drop in efficiency due to the increased head.)

4. 1000 gpm = 2.22 cfs

$$70(\text{eff}) = \frac{2.22(62.4)(1.2)(140)}{550}$$

$$\text{eff} = \underline{\underline{60.5\%}}$$

For uniform (constant depth) flow conditions in open channels, the Manning formula may be written in the form

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2}$$

When the cross-sectional area of flow is known, the formula may be solved directly for Q or S. If both Q and S are known and the cross-sectional area or depth of flow is desired, a trial and error solution based on the assumption of depth for uniform flow is generally the quickest.

Example: Depth of Fluid Flowing in Channel

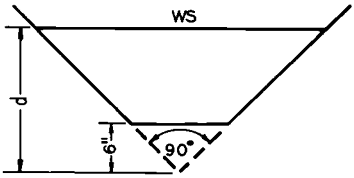
The Great Northern Paper Company of Maine has a log sluice 2800 feet long and built with a section, as shown. The sluice is built on a slope of 0.035 and capable of handling 125,000 fbm per hour when the flow of water is 200 cfs. If the channel roughness is taken at 0.020, what is the approximate depth d of the water in the flume?

$$Q = \frac{1.486}{0.020} AR^{2/3} (0.035)^{1/2}$$

$$Q = 13.9\ AR^{2/3}$$

$$A = d^2 - 0.25$$

$$\text{Wetted perimeter } P = 2.83d - 0.41$$

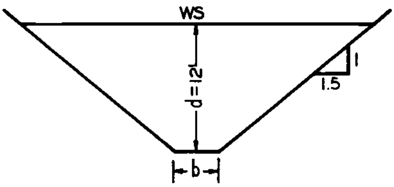


d (assumed)	A	P	R = A/P	$R^{2/3}$	Q
3	8.75	8.09	1.08	1.05	128
4	15.75	10.90	1.45	1.28	280
3.5	12.00	9.50	1.26	1.17	195

The depth of flow will be approximately 3.5 - 0.5 = 3.0 ft.

Example: Design of Channel to Carry a Given Flow

A channel is to be excavated that will carry a flow of 2500 cfs. The general slope of the country will permit a grade of only 0.0614% (S = 0.000614) and a water depth in the channel of 12 feet. Using a coefficient of roughness of 0.04 and side slopes of 1-1/2 to 1, determine the dimensions of the channel and the velocity of flow.



$$Q = \frac{1.486}{0.040} AR^{2/3}(0.000614)^{1/2}$$

$$Q = 0.92\ AR^{2/3}$$

$$A = 12b + 216$$

$$P = b + 43.3$$

b (assumed)	A	P	R	$R^{2/3}$	Q
10	336	53.3	6.30	3.42	1050
20	456	63.3	7.21	3.74	1570
40	696	83.3	8.36	4.11	2630
39	684	82.3	8.30	4.09	2570

Use a bottom width of 38 feet, then

$$V = \frac{Q}{A} = \frac{2500}{672} = \underline{\underline{3.72 \text{ ft/sec}}}$$

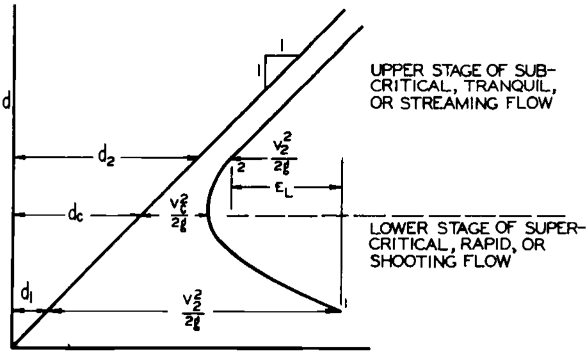
The dimensions shown are not necessarily the dimensions which give the most efficient cross section for the canal. The most efficient cross section is that which gives a minimum wetted perimeter. This minimum perimeter (maximum hydraulic radius) for a given area occurs when R = d/2 for any trapezoidal section, including a rectangle.

In the example shown above, a rearrangement of dimensions would permit the same velocity of flow on a flatter slope, or would permit a greater velocity and smaller area to be excavated on the same slope.

If water flows in a channel at some constant discharge, the velocity and depth of flow will, of course, vary with the slope of the channel. As the slope increases, the velocity will increase and the depth will decrease. The sum of depth plus velocity head at any point on the flow line of the channel is called specific energy, and may be written

$$E_s = d + \frac{V^2}{2g}$$

If this expression is plotted for different depths, assuming a constant discharge, a curve results, as shown.



$E_s = d + \frac{V^2}{2g}$; Q = CONSTANT

It is evident that at some point flow will occur with a minimum specific energy. This point is called critical flow, and flow is said to occur at critical depth and critical velocity. The channel slope which produces these conditions is called critical slope. Many hydraulic phenomena are tied to this idea of critical flow. The hydraulic jump, flow over a broad-crested weir or through a Parshall flume, and entrance control for flow through culverts are some examples.

In order for a hydraulic jump to occur, it is necessary that flow initially be at some depth lower than critical depth. If a rise in the downstream water surface occurs, the flow will then go through an abrupt transition from lower stage to upper stage, or from point 1 to point 2, with an accompanying energy loss, as shown.

Example: Grade Check for Hydraulic Jump

A wooden flume 6 feet wide and 4 feet deep is intended to carry water to a depth of 3 feet. What is the steepest grade on which this flume can be built to avoid the possibility of a hydraulic jump? Assume the flume is to be made of planed lumber where the value of roughness coefficient is 0.012.

If flow enters the lower stage, a jump can occur; therefore, critical slope is the limiting slope. For rectangular channels, critical flow occurs when the velocity head is equal to one-half the depth.

$$\frac{V^2}{2g} = \frac{d}{2} = 1.5 \text{ ft}$$

$$V = 9.85 \text{ ft/sec}$$

From the Manning formula

$$S_c = \left[\frac{Vn}{1.486\, R^{2/3}}\right]^2 = \left[\frac{9.85(0.012)}{1.486(1.5)^{2/3}}\right]^2 = \underline{\underline{0.0037}}$$

In the design of engineering projects it is often possible to arrive at two or more different solutions to a given problem. If each of these possible designs offers adequate performance, it is then necessary to compare them on the basis of cost. The best solution is the one which may be effected for the least annual cost over the life of the project. In this case, interest rates, operating costs, and the life of the project are important factors, in addition to the first cost.

Many plans of financing are available for any given project. For the purposes of design comparisons, however, the capital recovery factor is probably most useful. The use of this factor, which may be obtained from interest tables or computed approximately (CRF = 1.00/life of project + 2/3 interest rate), assumes a financing plan whereby first cost plus interest is written off in equal annual installments over the life of the project. The annual cost for financing the project is computed as the capital recovery factor multiplied by the first cost.

Example: Comparison of Pumping Costs

An industrial plant requires water at the maximum rate of 1000 gpm. The total pump life, including static and friction head, is 80 feet. The pump is driven by an electric motor, and power for the motor costs 1.5¢ per kwhr. The unit will operate under a load factor of 80%. Based on a life of 20 years, an interest rate of 5%, and equal maintenance costs, which of the following units would you buy? Show comparison.

	Unit A	Unit B
Motor efficiency	89%	88.5%
Pump efficiency	75%	72.0%
Cost	\$1200	\$1000

$$\text{hp to water} = \frac{QH}{8.8} = \frac{2.22(80)}{8.8} = 20.2 \text{ hp}$$
$$= 15.1 \text{ kw}$$

1 kwhr per year cost = 0.015(24)(0.8)(365) = \$105/yr

	Unit A	Unit B
Power cost = 15.1/eff (105)	\$2380	\$2490
Financing costs, CRF = 0.0802	96	80
Annual cost	\$2476	\$2570

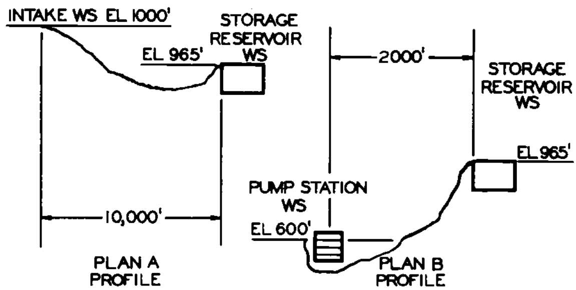
Unit A is most economical over a 20-year period, and will show increased advantage if the life of the unit is longer.

In the previous example the capital recovery factor was taken from interest tables. The approximate value would be

$$\text{CRF} = \frac{1.00}{20} + \frac{2}{3}(0.05) = 0.0833$$

Example: Comparison of Alternative Water Supply Systems

An investigation of the available means of expanding a water supply system has indicated that two methods are possible by which the required capacity of 2 mgd can be obtained. The two possibilities are Plan A and Plan B, as sketched.



Use either 10-, 12-, or 14-inch ID, 7 gage, asphalt dipped and wrapped pipe and assume the following costs will prevail. (These costs include all contractors' overheads, profit, et cetera.)

Trench excavation and backfill--all pipe sizes $0.90 per linear foot.

Cost of pipe in place in trench, complete:

10-inch ID	$3.60 per lin ft
12-inch ID	$4.90 per lin ft
14-inch ID	$5.60 per lin ft
Stream crossing for Plan A, complete	$4000 lump sum
Intake for Plan A, complete	$11,000 lump sum
Pump station for Plan B, complete	$26,000 lump sum

Connection to storage reservoir,
either plan, all pipe sizes,
complete $2000 lump sum

Assume further that the friction loss for the pipe is determined by C = 100 in the William-Hazen formula, or f = 0.025 in the Darcy formula.

Based on the above, determine the following:

1. Required pipe size and construction cost for Plan A.
2. Pumping head and construction cost for Plan B using 10-inch pipe.
3. Given annual pumping cost of $5700 for Plan B, find the total annual cost of financing and operating the two alternates, assuming
   a) Life of the facilities constructed is 30 years.
   b) Facilities will be financed by a 30-year bond issue at 3% interest, which bond issue will be retired by annual payments beginning the year after construction.
   c) Bond retirement schedule will be so set up that the annual payment for debt service, consisting of principal and interest, will be approximately the same for each year.
   d) Thirty per cent of the construction cost will be required for engineering, legal fees, and contingencies.
   e) Labor and material costs for operation, maintenance and repair will average $3000 per year for either plan.

$$2 \text{ mgd} = 3.10 \text{ cfs} = 1400 \text{ gpm}$$

1. Pipe size for Plan A:

$$h_L = (1000 - 965)/10 = 3.5 \text{ ft}/1000$$

Hazen-Williams, $\underline{\underline{D = 14 \text{ in.}}}$ (nomograph, Seelye)

Check by Darcy-Weisbach:

$$V = 2.90 \text{ ft/sec} \qquad \frac{V^2}{2g} = 0.13$$

$$h_L = \frac{25}{1000}\left(\frac{1000}{1.17}\right)(0.13) = 2.8 \text{ ft} \qquad \underline{\underline{14 \text{ in. OK}}}$$

For 12-inch:

$$h_L = 2.8\left(\frac{14}{12}\right)^5 = 6.1 \text{ ft} \qquad \underline{\underline{\text{12 in. too small}}}$$

2. Pumping head for Plan B (using 10-in. pipe):

Hazen-Williams $\quad h_f = \frac{18}{1000}(2) = \underline{\underline{36 \text{ ft}}}$

Darcy-Weisbach $\quad h_f = \frac{25}{1000}\,\frac{2000}{10/12}\,0.5 = \underline{\underline{30 \text{ ft}}}$

Pumping head = 365 + 35 = $\underline{\underline{400 \text{ ft}}}$

Construction costs:

Item	Plan A	Plan B
Trench excavation & backfill	\$ 9,000	\$ 1,800
Pipe in place	56,000	7,200
Stream crossing	4,000	--
Intake	11,000	--
Pump station	--	26,000
Reservoir connection	2,000	2,000
Total construction cost	82,000	37,000
Engineering, legal, et cetera at 30%	24,600	11,100
Amount of bond issue	106,600	48,100

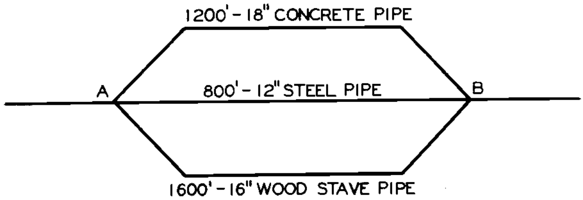
Annual costs:

Debt service at 5.1% (CRF)	5,450	2,460
Maintenance	3,000	3,000
Pumping costs	--	5,700
Annual cost	\$8,450	\$11,160

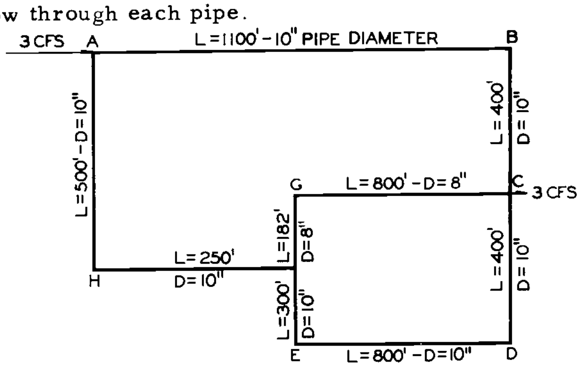
Problems

1. A horizontal rectangular conduit 2000 feet long by 8 feet wide by 10 feet high with invert elevation = 1000 feet msl, connects two reservoirs. A rectangular slide valve located 1500 feet from the upstream end of the conduit is open 2 feet and stands at elevation 1025 feet msl. Assuming the hydraulic energy losses through the valve consist of a gradual contraction equal to 0.1 the difference between the velocity head in the fully contracted jet (C_c = 0.6) and the conduit velocity head, and an abrupt expansion equal to 0.5 the same difference in velocity heads, determine the upstream pool elevation. The Darcy friction factor is f = 0.017, and in terms of the conduit velocity head, entrance and exit losses are 0.05 and 1.00, respectively.

2. When a quantity of 8000 gpm is flowing through a combination of pipes from A to B, as shown, what is the flow through each pipe, and what is the slope of the hydraulic gradient of each from A to B?



3. Water flows at the rate of 3.0 cfs through the pipe grid system shown, entering at A and flowing out at C. Determine the rate of flow through each pipe.



4. What is the depth of water flowing 1500 second-feet in a 40-foot channel with 2:1 side slopes? The gradient of the channel is 2 feet per mile and the roughness coefficient is 0.030.

5. The average annual duty of water for a 10,000-acre irrigation project is 3 feet. Estimate the distribution and maximum rate of use and the probable losses.

a) Design a main canal with moderate side slopes for construction through a light volcanic ash soil.

b) Design for the same canal a section with 30-degree side slopes across loose talus.

c) Design for the same canal a section with 30-degree side slopes in firm but seamy and fissured rock.

6. A municipal power system purchased annually 70,000,000 kwhr of electrical energy from Bonneville Power Administration at an average rate of 2.2 mills per kwhr. They understood the rate would be increased about 12% when the rates would be reviewed a year later. The city has water rights to 300 sec-ft on a stream where they have tentative plans for a power project which could provide this energy by developing a head of 180 feet. The proposed plan includes a diversion canal which is being designed for 300 sec-ft, but which will carry an average of only 250 sec-ft during the year.

A cost estimate of the plant is being made and, in the meantime, you have been asked to make a preliminary estimate of the maximum first cost the city would be justified in paying for the development. The project is to be financed by bonds maturing in 35 years and drawing 3-1/2% interest. Your estimate will be based on the following assumptions:

a) Transmission loss from plant to substation where BPA power is delivered is 3%.

b) Overall plant efficiency is 83%.

c) Operational maintenance and replacement are 1.5% of the project cost.

7. A multiple-purpose project is to be constructed at a total first cost of $30,000,000. Construction is to start at the beginning of the fiscal year and completed in five years. Funds will be expended at 5% the first year, 15% the second, 30% the third,

30% the fourth, and 20% the fifth. The capital cost, including interest during construction, must be amortized in 50 years from the date of completion. Interest rate is 4%, cost of operation and maintenance amounts to $200,000 annually, and loss of taxes $3000 annually. (Amortization rate at 4% is 4.655%.) Annual benefits amounting to $3,000,000 begin to accrue to the project upon completion. Determine the benefit-to-cost ratio for the project.

VI. HYDROLOGY

The hydraulic design of all drainage systems is based upon the peak flows which these systems must pass. For predicting peak flows for small areas the "rational" formula is widely used,

$$Q = CIA$$

where

- Q = peak runoff in cubic feet per second.
- C = runoff coefficient, which can be expected to vary with the size, shape, slope, and location of the watershed, as well as with topography, vegetation, soil texture, and soil moisture. The fact that this coefficient must be estimated makes judgment and experience important factors.
- I = rainfall intensity (inches per hour) taken from rainfall intensity-duration studies for a given area.
- A = area of watershed in acres.

<u>Example: Drain Pipe Sizes for Parking Lot</u>

An asphalt-paved parking lot is drained by catch basins located at A, B, C. The finished pavement elevations at these points are 55, 55, and 56, respectively. The inlet time for any inlet may be taken as 7 minutes, and the rainfall intensity-duration relationship is expressed as $I = 40/(t + 13)$. The minimum cover is 2.0 feet, and $n = 0.013$. What diameters of concrete pipe would you select for AC, BC, and CD?

Assume runoff coefficient $C = 0.90$. (Use nomograph in Seelye for pipe sizes.)

For area B:

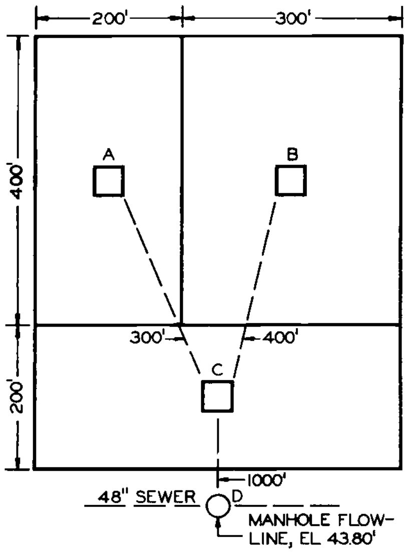
$$\text{Area} = 2.75 \text{ acres}$$

$$I = \frac{40}{7 + 13} = 2.0 \text{ in./hr}$$

$$Q = 0.90(2.0)(2.75) = 4.95 \text{ cfs}$$

Try D = 15-in. pipe: $V = 4.1$ ft/sec; $s = 0.006$; $h_L = 2.40$ ft.

Invert elevation at B = 51.75.



For area A:

$$\text{Area} = 1.83 \text{ acres}$$
$$I = 2.0 \text{ in./hr}$$
$$Q = 3.30 \text{ cfs}$$

Try D = 12 in. pipe: V = 4.3 ft/sec; s = 0.009; h_L = 2.7 ft

Invert elevation at A = 53.00.

For total area:

$$\text{Area} = 6.87 \text{ acres}$$

Time for flow to travel from B to C = 1.5 min approx

$$I = \frac{40}{8.5 + 13} = 1.86 \text{ in./hr}$$

$$Q = (0.90)(1.86)(6.87) = 11.5 \text{ cfs}$$

Try D = 24 in. pipe: V = 3.7 ft/sec; s = 0.0027; h_L = 2.7 ft

Invert elevation at C = 51.75 - 2.4 - 0.75 = 48.60

Invert elevation at D = 48.60 - 2.70 = 45.90

Top of 24-in. pipe at D = 47.90
Top of 48-in. pipe at D = 47.80
OK. Drop 24-in. pipe 0.1 at D

Where larger drainage areas are involved, the "rational" formula cannot be applied because of the complexities in estimating the runoff coefficient. In addition, it is generally desirable to know not only the peak flows but also the time distribution of runoff. A unit hydrograph constructed from actual stream flow records is useful in this case. Because the unit hydrograph is constructed from actual stream flow records, it is not necessary to estimate a runoff coefficient.

A unit hydrograph is defined as the hydrograph resulting from a unit storm which resulted in a volume of runoff equal to a depth of one inch over the entire drainage area. A unit storm may be of any length, but usually is taken as some even multiple of a 3-hour storm.

The use of the unit hydrograph in deriving design floods for a given drainage area is based on the following assumptions:

1. For all uniform-intensity storms of the same duration, the base length of the hydrograph is the same regardless of the volume of runoff.

2. Since the area under the hydrograph represents the total volume of runoff, the hydrographs of two unit storms of different intensities will be similar in shape to each other and to the unit hydrograph. The ratio of the ordinates of the graphs will be in the same ratio as the volumes of runoff from the storms.

3. The shape of the hydrograph is not changed because of concurrent runoff from another unit storm.

Example: Determination of Unit Hydrograph

A certain river has a watershed area of 75 square miles above an established recording gaging station. During a 6-hour rainstorm, 2 inches of rain fell evenly over the entire area, and then the rain stopped. The hydrograph of the stream, taken at 6-hour intervals, showed the following stream flow:

Time, hr	Observed flow, cfs
0	500
6	2440
12	3910
18	2630
24	1860
30	1480
36	1110
42	880
48	710
54	610
60	560
66	520
72	500

Determine the ordinates of the unit hydrograph for this stream and runoff coefficient for the 6-hour storm.

The solution may be outlined as follows:

1. Separate base flow of 500 cfs.
2. Compute volume of runoff resulting from storm.
3. Compute inches of runoff resulting from storm = R.
4. Ordinates to unit hydrograph = $\dfrac{\text{observed flow - 500}}{\text{R}}$
5. Runoff coefficient = R/2 = 1.39/2 = 0.695.

Time, hr	Storm flow, cfs	Unit hydrograph, cfs
0	0	0
6	1940	1395
12	3410	2450
18	2130	1530
24	1360	980
30	980	705
36	610	440
42	380	274
48	210	151
54	110	79
60	60	43

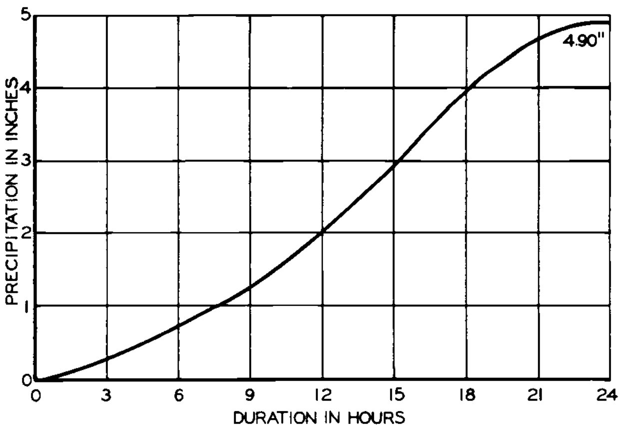
Time, hr	Storm flow, cfs	Unit hydrograph, cfs
66	20	14
72	0	0
	11,210	

Volume runoff = 11,210(6) = 67,300 cfs-hr

$$\text{Inches runoff} = \frac{67,300(3600)(12)}{75(5280)^2} = 1.39 \text{ in.}$$

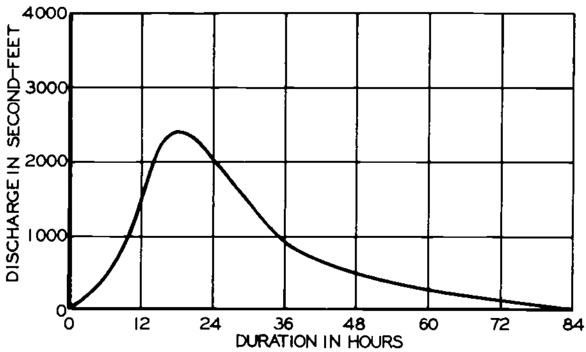
Example: Determination of Drainage Discharge

For the storm shown on the following mass precipitation curve, determine the maximum discharge at a bridge location which has a drainage area of 100 square miles of hilly country. A 12-hour unit hydrograph for the area above the bridge is given also. Surface losses during the storm were 0.08 inches per hour for the first 12 hours, and 0.05 inches per hour for the second 12 hours. Discharge at the bridge location at the start of the storm was 500 cfs.



In order to derive the storm hydrograph from the 12-hour unit hydrograph, it is necessary to use two 12-hour storms.

Runoff from first storm = 2.00 - 0.08(12) = 1.04 in.
Runoff from second storm = 2.90 - 0.05(12) = 2.30 in.



The storm hydrograph for the first storm is found by multiplying the ordinates to the unit hydrograph by 1.04. The storm hydrograph for the second storm, which starts 12 hours after the first, is found by multiplying the unit hydrograph ordinates by 2.30. The ordinates to these two hydrographs can then be added to obtain the total storm hydrograph.

Time hr	Unit hydrograph cfs	First storm cfs	Second storm cfs	Total cfs
0	0	0	0	0
6	600	625	0	625
12	1500	1560	0	1560
18	2500	2600	1380	3980
24	1900	1980	3450	5430
30	1300	1350	5750	7100
36	900	940	4370	5310
42	700	730	3000	3730
48	500	520	2080	2600

Maximum discharge = 7100 + 500 = 7600 cfs.

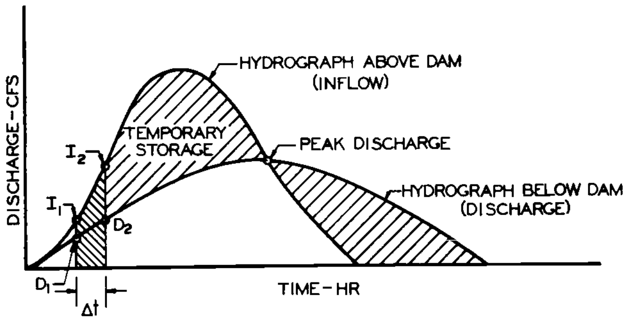
The effect of unregulated reservoir storage or pondage behind a dam during storm runoff is shown in the sketch. Note that an area on the graph represents a volume of water, and for some time interval Δt, the volume of water flowing into the reservoir is divided into two parts; one part discharged over the dam and one part held in storage. If subscripts 1 and 2 denote the beginning and end of the time interval, the relationship may be written

Volume inflow = volume discharged + volume stored

$$\frac{I_1 + I_2}{2} \Delta t = \frac{D_1 + D_2}{2} \Delta t + S_2 - S_1$$

$$I_1 + I_2 = D_1 + D_2 + \frac{2S_2}{\Delta t} - \frac{2S_1}{\Delta t}$$

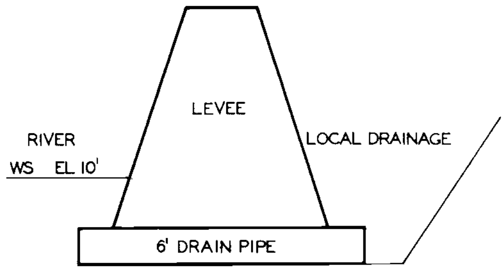
$$I_1 + I_2 + \left(\frac{2S_1}{\Delta t} - D_1\right) = \frac{2S_2}{\Delta t} + D_2$$



If discharge and storage are both a function of the water surface elevation behind the dam, a tabulation of water surface elevation versus $D + 2S/\Delta t$ can be constructed for some uniform time interval. Such time interval must be short enough to define the inflow hydrograph. Then starting at the initial point on the hydrograph for the first time interval, $I_1 + I_2 + (D_1 - 2S_1/\Delta t)$ can be found. From this value, which is equal to $D_2 + 2S_2/\Delta t$, the water surface elevation at the end of the first interval can be found. A step-by-step solution of the problem can then be effected.

Example: Determination of Water Surface Elevation During Storm

The storm runoff within a drainage district protected by a levee is passed through a 6-foot diameter pipe with a tide gate to the main river. The river, at the time of a local storm, is at a constant elevation of 10 feet.



The storm runoff from the local drainage behind the levee is defined by the following hydrograph:

Time, hr	Inflow, cfs
0	0
1	200
2	500
3	800
4	800
5	600
6	400
7	220
8	100
9	80
10	60

The channel storage behind the levee is at the constant rate of 10 acre-feet per foot of rise. The outflow through the culvert for each foot of head is given by the following table:

Head, ft	Outflow, cfs
0	0
1	191
2	270
3	330
4	381
5	426
6	467
7	505
8	539
9	572

Head, ft	Outflow, cfs
10	606
11	633
12	662

Assuming the water surface elevation inside the levee to be at 10 feet at the beginning of the storm, what is the maximum water surface elevation behind the levee that would result from the storm runoff?

Assume a time interval, $\Delta t = 1$ hr

$$\frac{2S}{\Delta t} = \frac{2(10)}{1} = 20 \text{ acre-ft per hr per ft of rise}$$

$$= \frac{20(43,600)}{3600} = 243 \text{ cfs per ft of rise}$$

A $2S/\Delta t + D$ table may now be constructed for $\Delta t = 1$ hour.

Head ft	D cfs	$2S/\Delta t$ cfs	$2S/\Delta t + D$
0	0	0	0
1	191	243	434
2	270	486	756
3	330	729	1059
4	381	972	1353
5	426	1215	1641
6	467	1458	1925
7	505	1701	2206
8	539	1944	2483
9	572	2187	2759
10	606	2430	3036

With this information the storm hydrograph may now be routed through the levee. Sample calculations are shown:

Time interval 0-1:

$$I_1 = 0;\ I_2 = 200;\ D_1 = 0;\ 2S_1/\Delta t = 0$$

$$I_1 + I_2 + \frac{2S_1}{\Delta t} - D_1 = 200 = \frac{2S_2}{\Delta t} + D_2$$

$$\text{WS elev} = 10 + 1\left(\frac{200}{434}\right) = 10.46$$

$$\frac{2S_2}{\Delta t} = 0.46(243) = 112; \quad D_2 = 200 - 112 = 88$$

Time interval 1-2:

$$I_1 = 200;\ I_2 = 500;\ D_1 = 88;\ 2S_1/\Delta t = 112$$

$$I_1 + I_2 + \frac{2S_1}{\Delta t} - D_1 = 724 = \frac{2S_2}{\Delta t} + D_2$$

$$\text{WS elev} = 11 + \frac{724 - 434}{756 - 434} = 11.90$$

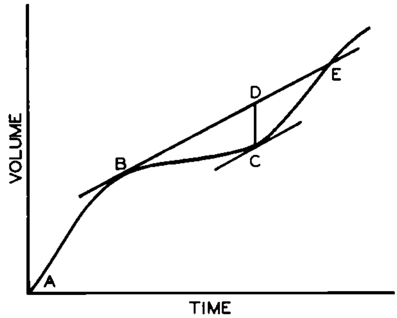
$$\frac{2S_2}{\Delta t} = 1.90(243) = 462; \quad D_2 = 724 - 462 = 262$$

Calculations for following time intervals are shown in the following table:

Time hr	Inflow cfs	$I_1 + I_2$	$\frac{2S_1}{\Delta t} - D_1$	$\frac{2S_2}{\Delta t} + D_2$	D	$\frac{2S}{\Delta t}$	Elev
0	0	0	0	0	0	0	10.00
1	200	200	0	200	88	112	10.46
2	500	700	24	724	262	462	11.90
3	800	1300	200	1500	400	1100	14.51
4	800	1600	700	2300	520	1780	17.34
5	600	1400	1260	2660	560	2100	18.64
6	400	1000	1540	2540	540	2000	18.21
7	220	660	1460	2120	490	1630	16.69
8	100	320	1140	1460	400	1060	14.37
9	80	180	660	840	275	565	12.28
10	60	140	290	430	190	240	10.99

Note that the maximum water surface elevation of 18.64 feet inside the levee occurred at least an hour after the peak runoff had passed, and that the peak outflow was 560 cfs against a peak inflow of 800 cfs.

Many engineering projects are concerned with minimum stream flows as well as maximum stream flows. Some examples of these are power generation, water supply, irrigation, and navigation. Here again, stream flow records are referred to and are examined with the idea of increasing minimum flows by the use of storage. The graphical approach of the mass diagram is most useful in this study. Cumulative volumes of flow past the site of the development are plotted against time (see below).



Note that the slope of a line on this diagram represents a volume rate of flow. If we say that BDE represents some minimum desired flow rate, we see that this flow rate is exceeded during times AB and CE. The natural flow is less than that desired during time BC. If the reservoir is full at A, we will be spilling water through AB; drawing from storage through BC; and refilling the reservoir through CE. After time E, we will be spilling water again. The length of line DC represents the volume of storage necessary to develop the desired flow.

Example: Determination of Minimum Flow

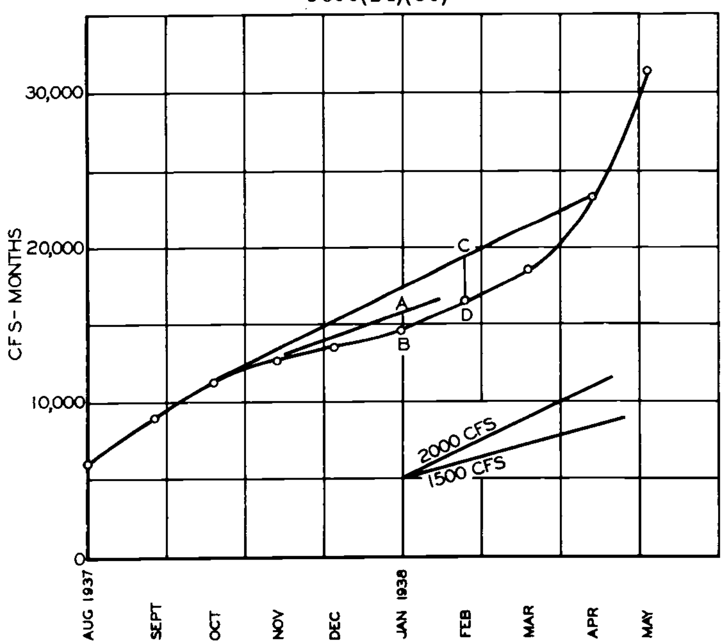
Determine maximum dependable flow at a power dam which forms a reservoir with a gross capacity of 180,000 acre-feet, of which 120,000 acre-feet are usable and the critical power period of the record of the stream at the dam is

Date	Mean monthly discharge, cfs
1936 June	9,000
" July	15,000
" Aug	12,000
" Sept	7,000
" Oct	6,000
" Nov	3,000
" Dec	1,500
1937 Jan	1,200
" Feb	1,200
" Mar	2,000
" April	2,500
" May	4,500

Date	Mean monthly discharge, cfs
1937 June	5,500
" July	8,000
" Aug	6,000
" Sept	3,000
" Oct	2,000
" Nov	1,500
" Dec	1,100
1938 Jan	1,000
" Feb	1,500
" Mar	2,500
" April	5,000
" May	7,000
" June	10,000
" July	16,000
" Aug	12,000

Plot a mass diagram for the cumulative flows, as shown.

$$120{,}000 \text{ acre-feet} = \frac{120{,}000(43{,}600)}{3600(24)(30)} = 2000 \text{ cfs-month}$$



A table of required storage versus dependable flow can be derived from the mass diagram.

Flow cfs	Storage cfs - mo	Line
1000	0	--
1500	1100	AB
2000	2700	CD

$$\text{Approximate dependable flow} = 1500 + 500\left(\frac{2000 - 1100}{2700 - 1100}\right)$$

$$= \underline{\underline{1780 \text{ cfs}}}$$

Problems

1. Evaluate the following equivalents to three significant figures:

a) 1 - day - cfs = (?) acre-ft
b) 1 - month - cfs = (?) acre-ft (31-day month)
c) 1 - year - cfs = (?) acre-ft
d) 1 - acre-ft/sq mile = (?) inches of runoff
e) 1 - cfs - year/sq mile = (?) inches of runoff
f) 1 - cfs = (?) million gallons per day
g) 1 inch of runoff per day per sq mile = (?) cfs
h) 1 inch of runoff per hr per acre = (?) cfs

2. a) Explain how to construct a unit hydrograph used in routing river flood crests.

b) Draw and explain how to construct a rainfall intensity-duration curve. Explain how this curve is used in engineering practice.

c) Are water rights required for underground waters in Oregon, and if so, to whom would one have to make application for such use?

3. Determine the required reservoir capacity to give dependable regulated flow of 950 cfs for the stream flow pattern given. Also determine for this same period of record the dependable flow that can be obtained with 430,000 acre-feet of usable reservoir capacity.

April	2000 cfs month	May	2000
May	3000	June	3000
June	4500	July	2500
July	3500	Aug	1800
Aug	2500	Sept	1200
Sept	1500	Oct	900
Oct	1000	Nov	800
Nov	800	Dec	700
Dec	750	Jan	600
Jan	700	Feb	750
Feb	800	Mar	1000
Mar	1100	April	1500
April	1500	May	2500

VII. SANITARY

In the design of water supply systems the problems of quality and quantity of water are most important. Specifications for water quality are set by the U.S. Public Health Service Drinking Water Standards, February 1946, and specify permissible limits as to bacteriological quality as well as physical and chemical characteristics. Required quantity, assuming an adequate supply, is based on estimated per capita use, industrial demand, and any other unclassified use. Design flows for a given project cannot be estimated safely until a detailed study is made of water use for that project.

If an adequate source of supply protected from pollution can be developed, the next steps in the process are treatment of the water in order to make it pleasant to the taste, sight, and smell; removal of disease-producing bacteria; and reduction of excessive concentrations of certain chemicals. Treatment may be given by any one or combination of the following:

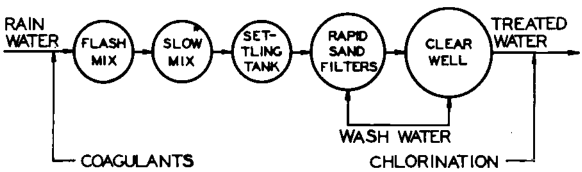
1. Sedimentation--with or without coagulants, to reduce turbidity, taste, and odor.

2. Chemical treatment--to reduce hardness or change pH in order to render water less corrosive, or to implement sedimentation.

3. Filtration--to reduce bacteria count, turbidity, taste, and odor.

4. Disinfection--chlorination to reduce bacteria count.

A flow diagram for a typical water treatment plant employing a rapid sand filter is shown below.



The purpose and the basis for design of each of the units shown are outlined below:

1. Flash mix--thoroughly mixes coagulants with the raw water. This may be accomplished in a number of ways: jet-agitated mixing basin; mechanical mixer (pump, paddle wheel, etc); baffled basin, spiral flow basin, or hydraulic jump. Design criterion is thorough mixing, usually accomplished in 30 to 60 seconds' detention time.

2. Slow mix--flocculator--allows the floc formed by the reaction of coagulants with the water to form. A mechanical agitator traveling at slow speeds is used. Design criterion is optimum floc formation, and design is based on a detention time of 20 to 40 minutes.

3. Settling tank--clarifier--allows sedimentation of floc. Design should include some means of sludge removal. Design is based on detention time of 2 to 6 hours, with overflow rates of 500 to 800 gal per sq ft per day.

4. Filter--removes bacteria, colloidal matter, and fine suspended solids. Design should include an under-drain system of perforated pipes or porous plates, approximately 18 inches of graded gravel if pipes under drains are used, and approximately 30 inches of filter media (quartz sand or crushed anthracite). Provisions for backwashing the filter through the under drains should be made also. Design is based on a filtering rate of 2 to 3 gal per minute per sq ft, and a backwash rate of approximately 15 gal per minute per sq ft.

5. Clear well--storage for filtered water. Volume of the clear well depends upon other storage available in system. Commonly used figures are 1/4 to 1/3 daily plant capacity.

Example: Design of Water Treatment Plant

A city with 50,000 population desires to build a water treatment plant to supply water from a river that runs nearby. It is estimated the consumption of water will be approximately 120 gal/capita/day, and industrial consumption will be 5 mgd. Assume the raw water has no appreciable odor, only a slight color, a turbidity of 30 ppm, total hardness of 20 ppm, total alkalinity as

$CaCO_3$ of 30 ppm, and a pH of 7.2. A total count of bacteria shows a high concentration and indicates the need of a rapid sand filter.

1. In a preliminary estimate,
   a) What chemical would you recommend be used?
   b) How would you add the chemicals?
   c) What method of rapid mixing would you use?
   d) What should be the size of the coagulation basin?
   e) What should be the size of the sedimentation basin?
   f) What should be the area of the filter beds?
   g) What kind of filter media would you use?
   h) What size clear well should be used?

2. Draw a schematic diagram of the plant and label all parts.

This water falls within USPHS Standards except for turbidity, which must be removed by use of coagulants.

Design flow = 5 + 6 = 11 mgd = 17.1 cfs.

1a) Use aluminum sulphate (alum).
1b) Dry feed.
1c) See previous discussion.
1d) Assume 30 minutes detention time

Vol tank = 17.1(60)(30) = 30,800 cu ft
Use depth = 15.8 ft
Dimensions = 25 by 78 ft (may be divided)

1e) Assume detention time of 4 hours and overflow rate of 700 gpd per sq ft

Vol tank = 17.1(3600)(4) = 248,000 cu ft

$$\text{Surface area} = \frac{11,000,000}{700} = 15,700\ \text{ft}^2$$

Depth of tank = 15.8 ft
Dimensions = 25 by 625 (may be divided)

If a shorter detention time is permissible, smaller dimensions may be adopted.

1f) Assume filter rate of 2 gpm/sq ft

$$\text{Area} = \frac{11,000,000}{2(60)(24)} = 3800\ \text{sq ft}$$

1g) Use 30 inches of clean quartz sand--effective size of 0.40 to 0.55 mm and uniformity coefficient of approximately 1.5.

1h) Assume 1/4 daily capacity.

The decomposition of sewage wastes in the natural processes of decay requires oxygen from an outside source if disagreeable odors and pollution of streams and lakes are to be avoided. The need for oxygen is defined as the biochemical oxygen demand (BOD) of the sewage and is expressed in parts per million (ppm) by weight.

Sewage treatment processes may be grouped in three general classes:

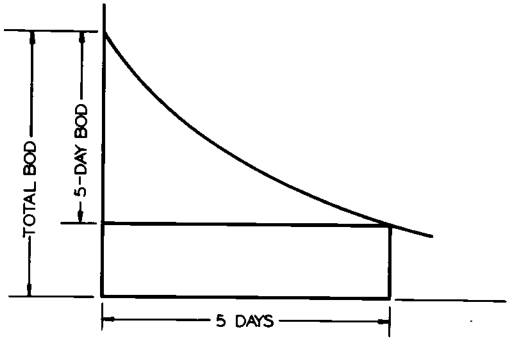
1. Dilution--direct discharge of wastes into natural bodies of water. The BOD is satisfied by means of dissolved oxygen (DO) in the stream, or by additional oxygen supplied from the atmosphere through reaeration. If sufficient natural oxygen is not available, disagreeable odors will be present and fish life will be destroyed.

2. Primary treatment--removal through sedimentation and separate digestion of solids causing BOD, with the untreated effluent being disposed of by dilution. Overall removals are 40 to 70% of suspended solids and 30 to 40% reduction of BOD.

3. Secondary treatment--includes primary treatment with further treatment of the effluent by filtering. Overall removals are 80 to 90% of suspended solids and 90 to 95% reduction of BOD.

If oxygen is furnished to sewage the natural demand for oxygen will decrease as the processes of decay proceed along the lines shown in the sketch.

The 5-day BOD is normally taken as the index of the oxygen requirements for the sewage. The percent stability of a sample is defined as the percentage of the total BOD used since the beginning of the test. A stability of 68% at the end of 5 days is considered to be an average value.

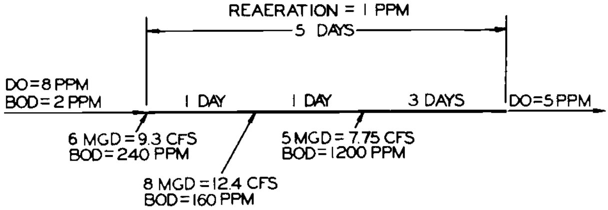


Example: River Flow Necessary to Maintain Minimum Dissolved Oxygen

At a certain point in a river the dissolved oxygen (DO) is 8.0 ppm and its BOD is 2.0 ppm. At this same point there are 6 mgd of domestic sewage with a BOD of 240 ppm discharged into the river. One day downstream from the initial point, 8 mgd of domestic sewage with a BOD of 160 ppm enters the river, and two days downstream from the initial point, 5 mgd of industrial wastes with a BOD of 1200 ppm enters the stream. Assume that reaeration of the river equals 1 ppm in 5 days, and that all BOD values are 5 days at a temperature of 20°C, and all entering sewage wastes have zero DO. Given data of BOD:

Days	% Stability
1	21
2	37
3	50
4	60
5	68

What flow in cfs would be required at the initial point to maintain 5 ppm of dissolved oxygen 5 days downstream from the initial point?

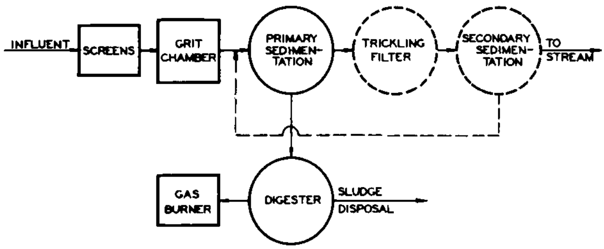


Oxygen demanded by BOD = dissolved oxygen available.

$$9.3(240) + 12.4(160)\left(\frac{60}{68}\right) + 7.75(1200)\left(\frac{50}{68}\right) = Q\ (8 + 1 - 2 - 5)$$

$$Q = \underline{\underline{5410 \text{ cfs}}}$$

A flow diagram for a typical sewage treatment plant is shown below. Dotted lines refer to units which give secondary treatment.



The purpose and basis for design of each of the units shown are outlined below:

1. Grit chamber--to remove sand and gravel by sedimentation. Design is based on a carrying velocity of 0.5 to 1.0 ft per sec, and a detention time of 30 to 60 sec.

2. Primary sedimentation--to remove settleable solids causing BOD. Design is based on a detention time of 1 to 2 hours, and overflow rate of 600 to 1200 gal per sq ft per day. Tanks should be provided with scum removal and sludge collection devices.

3. Digester--to provide space for sludge digestion by means of anaerobic bacteria. Should be provided with gas vent,

means of drawing off digested sludge, and heat coils (80 to 100°F). Design is based on 2 to 4 cu ft per capita, or on a solids loading basis.

4. Trickling filter--to satisfy BOD of primary effluent. Design is based on a filter loading of approximately 400 lb of BOD per acre-ft per day for a standard rate filter, and 1.5 to 2.0 lb of BOD per cu yd of filter rock per day.

Example: Primary Treatment Plant Design

A city of 10,000 population has a sewage flow of 2.0 mgd. After considerable study of the sewage quality, location for the desired sewage disposal plant, disposal of effluent, state and local health requirements, et cetera, the design of a primary treatment plant has been decided upon. It is thought desirable to plan on a plain settling basin and sludge digestion system consisting of primary and secondary digestion tanks.

You are asked to determine the size of the settling basin and the necessary size of the digestion tanks. Basic design data include:

Raw sewage, 2.0 mgd
Allowable overflow rate, 600 gal/sq ft/day
Detention time, 1.5 hr
Suspended solids, raw sewage, 300 ppm
Required removal, 60% of suspended solids
Recommended loading, 5 lb/cu ft/mo for primary digester
Digesters to be designed on dry solids loading basis

Make a sketch of the plant showing not only the settling tank and digesters, but other principal structures and apparatus that go with a primary treatment plant.

Volume of tank = 2(1.55)(3600)(1.5) = 16,700 cu ft, determined by detention time.

Surface area $= \dfrac{2,000,000}{600} = 3330$ sq ft, determined by overflow rate or surface settling rate.

$$\text{Depth of tank} = \frac{16,700}{3330} = 5.0 \text{ ft above sludge storage.}$$

$$\text{Diameter of tank} = D = \sqrt{A/0.785}$$

$$= \underline{\underline{65.0 \text{ ft}}}$$

An increase in overflow rate, if permissible, would allow revision of dimensions.

For digester:

$$\text{Weight of solids} = 2(300)(8.35)(0.6) = 3000 \text{ lb/day}$$

$$= 90,000 \text{ lb/mo}$$

$$\text{Volume of digester} = \frac{90,000}{5}$$

$$= 18,000 \text{ cu ft}$$

Assume a depth of 24 ft
Diameter = 31 ft

Example: Trickling Filter Design

Determine the dimensions of a high-rate trickling filter required to reduce 5-day BOD by 70% for raw sewage influent under the following conditions:

Population, 3000
Average daily domestic sewage, 70 gal/capita
5-day BOD (influent to filter), 210 ppm
Flow = 0.21 mgd
BOD load to filter = 0.21(210)(8.35)(0.70) = 258 lb/day

Assume a filter loading of 1.5 lb of 5-day BOD per cu yd per day.

$$\text{Volume of rock} = \frac{258}{1.5}(27) = 4650 \text{ cu ft}$$

Assume depth = 6 ft

Diameter = 31.5 ft

Problems

1. Draw a flow diagram of a rapid sand filter water treatment plant showing the places of application of the chemicals, if used.

a) What is the usual thickness of sand bed?
b) What is a reasonable rate of filtration?
c) What is a reasonable rate of backwash?
d) What should be the detention period in the settling basin?
e) What percentage of bacterial removal can be expected without chlorination?
f) What chemical is most used as a coagulant?
g) What other chemicals are in common use as coagulants?
h) Approximately how much coagulant must be used to get good floc?
i) Approximately how much chlorine is used for chlorination?
j) Why is ammonia sometimes used with the chlorine?

2. a) What quantity of sewage may need to be handled in 24 hours, and what dimensions should be used for a simple septic tank for 10 persons?

b) A sewage treatment plant has a flow of 2.1 million gallons of sewage daily during peak flow, at which time the BOD of the effluent is 50 ppm. The stream into which the plant discharges contains 6.2 ppm of dissolved oxygen when it is flowing at minimum stage of 25 cubic feet per second. What will be the residual dissolved oxygen in the stream just below the sewage outfall when maximum sewage flow is discharging into minimum stream flow?

c) If a sewage flow of 2 million gallons daily is to be prechlorinated by use of a bleaching powder containing 35% available chlorine, how many pounds of bleach will be required if the sewage influent is to receive a 5 ppm chlorine dosage?

3. Plain sedimentation is an important feature of two primary sewage treatment plants each having a design flow of 8 mgd and a detention time of 120 minutes. Plant A has 300 ppm suspended solids in the influent and an efficiency of 60% in the removal of suspended solids. At plant B, where the influent suspended solids

amount to 200 ppm, the efficiency in removal of suspended solids is 50%.

a) Which plant will discharge the least amount of suspended solids to the receiving stream?

b) What will be the total volume in cubic feet of the sedimentation tanks at plant A?

c) If a surface settling rate of 800 gallons per square foot per day is desired, what is the necessary side water depth of the sedimentation tanks at plant B, neglecting sludge collecting and storage space?

4. a) A community of 2500 population is building a sewage treatment plant whose final effluent is to have a suspended solids content of not more than 40 ppm and a 5-day BOD of not over 30 ppm. Assuming the necessary reasonable values of quantity and character of sewage to be treated, give the dimensions, retention time, and effluent characteristics (BOD and suspended solids) of the primary sedimentation tank.

b) The effluent of a sewage treatment plant has a BOD of 46 ppm, while the BOD of the same plant with less complete treatment is 25 ppm. Explain the cause of this condition.

5. A sewage treatment plant is to be constructed with a design basis of 5000 population and a sanitary low water flow of 0.75 mgd. Discharge is into a fairly large river a short distance from the ocean. What will be the various treatment units to be provided and their approximate dimensions?

6. a) What are the advantages and disadvantages of a "Downes" or floating cover for the digestion tank of a sewage treatment plant?

b) Name three general types of pumping devices used for sewage sludge, and briefly discuss the field of use for each.

c) What is the practice and what factors are to be considered in placing in operation a new digestion tank at a sewage treatment plant?

VIII. MISCELLANEOUS PROBLEMS

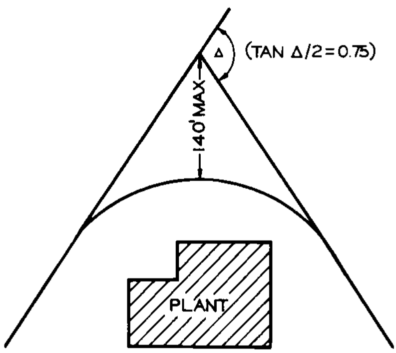
1. a) From the following field notes for a 30-foot flat roadway having cut slopes of 2 to 1, compute the volume of cut between the two stations by the prismoidal formula.

Station 102	+ 00	+ 7.5	+ 7.8	+ 5.5	+ 9.2	+ 7.1
		30.0	14.1	0.0	12.5	29.2
Station 103	+ 00	+ 6.2	+ 6.1	+ 4.0	+ 8.7	+ 10.2
		27.4	13.0	0.0	17.2	35.4

Would the prismoidal formula normally be used on these roadway sections? Why?

b) Two tangents of a highway intersect in the vicinity of a large industrial plant, as shown in the sketch. You want to connect the two tangents with the largest radius curve possible, but the curve must pass between the plant and the intersection. To allow an adequate right-of-way the external distance between curve and intersection cannot exceed 140 feet.

What degree of curvature (to the nearest even degree) of a simple curve will best meet these requirements?



2. A column is to be loaded to 100 tons. Foundation borings and tests have given the following information:

Depth, ft	Properties of soil
0 to 5	Clean, medium sand; allowable bearing of 4 tons per sq ft. Angle of internal friction, $\phi = 30$.
5 to 100	Clay; allowable bearing of 1 ton per sq ft.
100	Ledge rock.

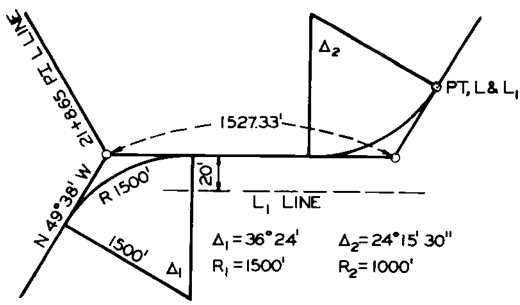
a) What is the required area of a square footing if placed on the surface of the sand?

b) What is the required area of a square footing if placed 2 feet below the surface of the sand?

c) The contractor proposes a footing supported on 4 wood piles 50 feet long, spaced on 2-foot centers. A load test on one pile gave an allowable load of 22 tons. Would you consider the pile foundation better than the footing in part a)? Explain reasons for your decision.

d) In the design of a large building, what methods may be used to minimize damage from differential settlement?

3. a) After a highway location has been completed, it is found desirable to make a 20-foot parallel shift on an intervening tangent. Given the data on the L line, establish all necessary transit data for the L_1 line. First curve to maintain R = 1500 ft and second curve to have same location for PT.



b) Sketch in profile the centerline of an ungraded road approximately 1000 feet in length, indicating a proposed finished subgrade thereon. Project beneath it a mass diagram illustrating cut, fill, shrinkage, overhaul, free haul, borrow, and waste.

4. Concrete aggregate, cement, reinforcing steel, and other construction materials must be trucked from a railroad siding for a distance of 10 miles over a single-lane road through sandy desert country. The trucks must meet and pass at definite points along the route where the road will be widened to accommodate two lanes of traffic. The loaded truck speed will be 10 miles per hour, and unloaded 20 miles per hour. The schedule for loading and unloading with a single crew available at each end is 15 minutes.

a) Determine how many trucks can be operated efficiently.
b) Locate the passing lanes.
c) Will there be any advantage in operating the trucks in pairs?
d) How many hours a day (trucks to work 8 hours as nearly as possible) will the loading and unloading crews be engaged if the trucks are operated singly?

5. Show computations on the following:

a) One cu ft sand with 40% voids weighs 110 lb. What is the weight in water?
b) One cu ft of sand received weighs 98.6 lb; dried, 89.1 lb. One cu ft of dry sand of this sample weighs 105.4 lb. What is the percentage of swell of the sand received?
c) Aggregate 1 - 70% passes the 10 mesh. Aggregate 2 - 40% passes the 10 mesh. What percentage passing the 10 mesh will give a combined mix of 40% of aggregate 1 and 60% of aggregate 2?
d) One cu ft of earth weighs 110 lb; its specific gravity is 2.62. What percentage of voids does it contain?
e) If the water-cement ratio by weight is 0.585, what is percentage of water per sack of cement?

f) How does moisture content of concrete affect the strength of concrete cylinders?

g) What is typical time of initial and final set as shown on cement test reports?

h) What kind of sand is used in making cement tests?

i) How is the presence of organic matter in concrete aggregate determined?

j) What percentage of cement passes the 100 and 200 mesh sieves, as shown by test reports?

6. A property with an investment of $8,000,000 has an average net revenue available for dividends of $320,000 after all charges have been paid. The financing plan resulted in the issuance of $4,500,000 in bonds, with average interest 3% and the remainder in common stock. Find the result to the stockholders.

7. a) Enumerate briefly the PRA designation and chief characteristics and suitability of each type of soil pertaining to highways, airfields, and dams.

b) What measures should be taken to stabilize the subgrade of a heavy traffic road to be built under low tideland conditions where mainly peaty-silt is encountered? Which class of material would be most suitable for stabilization?

c) In determining the site of a proposed airfield where flexible bituminous type runways and taxiways are to be built, what soil types would you seek?

d) When cement concrete is not available for a core wall of an earth dam, what types of soil should be selected for use and how should they be compacted? Show cross section of suggested dam with different types of materials used for core, for fill, and for surfacing, et cetera, with slopes indicated.

8. How much super elevation should be provided in constructing a new concrete roadway 20 feet wide on a main highway where there is a 2-degree curve, and the maximum speed for design purposes is 40 miles per hour?

9. a) It is proposed to pave a main highway with a concrete having an ultimate compressive strength of 2500 lb/sq in. If the Oregon Motor Vehicle Laws specify the maximum wheel load of 8500 pounds, what should be the thickness of the

pavement slabs? What should be the edge thickness of the slabs to avoid breaking off the corners? What water-cement ratio would you recommend for this concrete?

b) What is the difference between a bituminous concrete pavement and a bituminous macadam pavement?

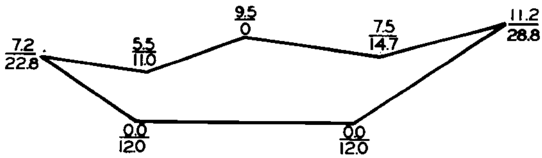
c) A vertical parabolic curve is to be used as a transition between a minus 3% and a plus 2% grade. The length of the curve is 800 feet. If the elevation of the PC is 496.34 feet, and the station of the PC is 8 + 00, find the elevation of 12 + 00.

10. a) Give two methods of estimating depreciation and set up a numerical comparison of the operation of each method for an article having a life of 10 years. Make reasonable assumptions for the other factors.

b) In a purchase and sale case, which method of depreciation is likely to be advocated by the seller, and why?

c) Which method most closely approximates the actual reduction of value experienced with articles commonly bought and sold, such as an automobile and a house? Explain why.

11. a) Calculate the area of the cross section shown below.



b) What is a "mass diagram," and how is it made?

c) Explain the theory of the stadia. Illustrate the application with a sketch.

12. A minus 1.8% grade on a highway meets a plus 2.2% grade at station 12 + 50, the elevation of the point of intersection being 863.2. If a vertical curve 600 feet long is required, what is the grade elevation of each station and half station?

13. The following paragraphs have been included in the general provision of certain engineering contracts, under which a

portion of the work may be done by subcontract. Discuss each paragraph. Is it a proper and satisfactory provision? What changes would you make to render it more equitable and satisfactory?

a) All extra work done shall be paid for at actual cost plus 15%.

b) The engineer shall be permitted to remove such portions of the work as he may from time to time think necessary for the discovery of improper material or workmanship, and the contractor shall restore such work at his own expense.

c) If the contractor should fail or refuse to execute the work properly and in accordance with the specifications, the engineer, after giving 30 days written notice, may take over the work.

14. A highway location required that a curve be located to pass through a specified point. The point was located by angle and distance from the PI of the curve. The engineer turned an angle of 28°10' to the left from the back tangent with the instrument set on the PI. He measured 126 feet from the PI to the point. The deflection angle between the back and forward tangents is 44°20' to the right.

Determine the radius R, degree D, tangent distance T, and the length L of the curve passing through the point.

15. Make computations and prepare complete set of transit notes necessary to place the located centerline for a highway curve, including spirals, with the following information:

Degree of central curve = 2°
Total delta angle = 30°32'
Spiral length (10 chord) = 400 ft
PI station = 12 + 00

Outline procedure for staking curve in field.

16. A 48-inch diameter culvert is to be placed in a trench 6 feet wide and 20 feet deep to invert elevation from the finished highway grade. ASTM specifications give 8000 pounds load per foot to produce a 0.1-inch crack for the 3-edge bearing test, and 1.5 times that for sand bearing for this 48-inch culvert pipe.

Determine the load per foot of pipe and the factor of safety for (1) ordinary bedding, and (2) standard concrete cradle bedding.

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